WATER POWER

A technical journal devoted to the study of all aspects of Hydro-electric developments

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Editor : H. STOREY, M.C., B.Sc., M.I.M.E.

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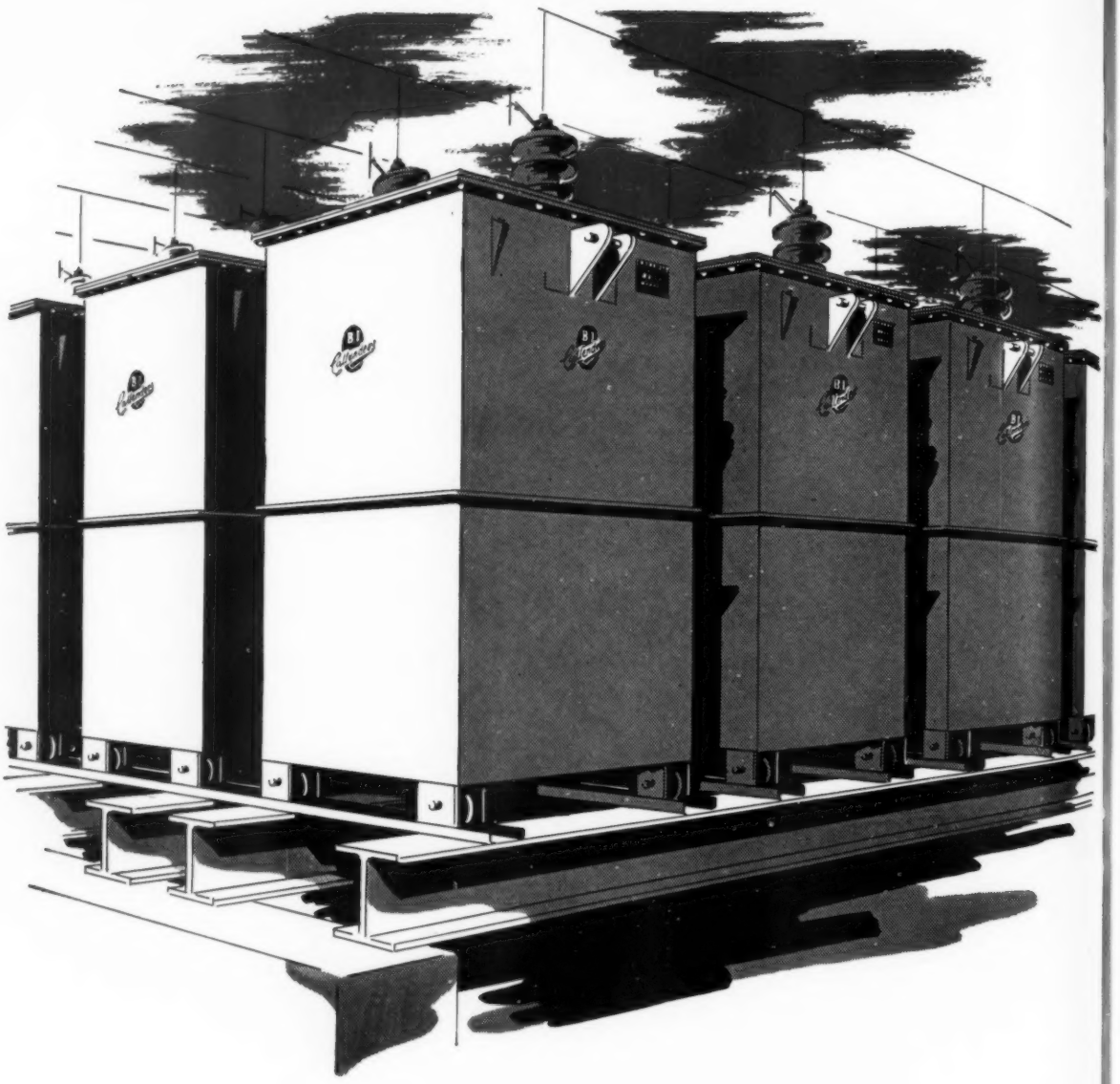

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WATER POWER

A technical journal devoted to the study of all aspects of Hydro-Electric Development

Economy of Effort

OF all power-producing engineers the hydro-electric specialist probably makes the least demand on the technical manpower and material of his country per unit delivered, either for the construction of his projects or for operating them when in service. If this is true, and we propose below to adduce evidence to support this view, the water-power engineer is performing a most valuable service, since the shortage of technicians of all grades, from the high-level design specialist to the skilled manual worker, is a problem which increasingly besets every industrial country.

In many walks of life, in these days, attention is devoted at the highest pitch to the devising of automatic procedures and machinery to fulfil the twin functions of economising in manual effort and of securing a more rapid accomplishment, in a generally more efficient manner, of the end in view. In the construction of hydro-electric dams, there is no less a focus of effort in this direction. The figures of man-hours required per cubic foot of completed structure, type for type, have shown a steady reduction over the years since the war, while the tasks demanded of those who work on such schemes have grown rapidly less arduous. For example, the use of such machines as diesel-driven graders with power-assisted steering and hydraulic operation of the major parts of the device, together with the provision of comfortable seating for the driver who is protected from the weather and kept warm in cold climates, is a typical instance of progression in this field.

The use of the "brute-force" philosophy for the shifting of earth has given way to a scientific approach whereby earth-moving and concrete-placing machinery nowadays requires less power per unit of work done (and consequently less fuel) and frequently carries out the job more efficiently.

Scientifically planned maintenance schedules, employing the services of ever more highly skilled technicians to keep the electrical and diesel power equipment in perfect trim at all times, means that construction delays are minimised and—at the expense of the provision of duplicate equipment, in many cases to allow a spare part to be instantly substituted for a defective element—there are virtually no hold-ups, and no possibility of being prevented from meeting programme dates by unexpected faults in the equipment.

The variety of earth-moving equipment available to the civil engineer nowadays is such as to provide him with almost the exact tool required for any conceivable purpose; for example, drag-line machines nowadays are made up in the form of lorry-mounted superstructures carrying a crane which can operate as a drag-line with a bucket of five-eighths of a cubic yard capacity. The machine can, of course, be used as a crane, having in this particular case a standard boom length of 35 ft., and a lifting capacity of 5 tons. The whole device is not only adaptable to many purposes, but is also extremely mobile, and therefore no time is wasted in dismantling, loading and re-erecting.

Many hydro-electric projects first considered perhaps 30 or 40 years ago were not proceeded with because it was thought that the construction work would be prohibitively expensive, if not technically impossible. The availability of modern machinery means that a different viewpoint can be adopted towards marginal schemes since it may well be practicable nowadays to construct such projects where even 10 years ago this would not have been the case.

In regard to tunnelling the constant increase in drilling speeds, using full-face tunnelling methods, has been made possible by improved machinery, resulting in economy of labour as well as extra speed, and thus again the situation may arise where a marginal project involving tunnelling has now become economically feasible.

Alone among the large-scale power-production plants hydro stations can be made to be almost automatic in operation, and indeed many are in fact completely automatic, and are operated in day-to-day running without a single attendant. The steady evolution of electronic gear of all kinds enabling completely re-

liable supervision to be exercised over complex generating plant, with reliable remote control from great distances, may well act as an incentive to develop hydro schemes in those countries where, at present, it is becoming extremely difficult to obtain men to staff the other types of power plant. It has, of course, always to be realised that the operating side of power supply is not the type of work which appeals to every engineer. There is in it, with the modern highly reliable plant, an element of monotony, and a lack of outlets for the creative energy of the engineer concerned.

For these reasons, it may well be held that in the power-production picture as a whole the hydro-electric engineer at present is in a position to use less of the precious skilled man-hours in construction, and far less man-hours per annum in operation, than is the case than any other form of power-production equipment.

New World Tunnelling Record

A NEW world tunnelling record is claimed to have been established during operations on the St. Fillans Section of the North of Scotland Hydro-Electric Board's Beadalbane project in Perthshire. In the seven working days—October 19 to 26—tunnelling crews of the Mitchell Construction Company, Peterborough, drove 557 ft. of 8 ft. 6 in. bed width through hard epidiorite rock. The previous world record was 551 ft. set up in the United States. The Mitchell Construction Company achieved a British and European record of 428 ft., also at Breadalbane, last December, though this was subsequently bettered.

The tunnelling crew, which consisted of three teams of 13 men each, worked in three eight-hour shifts round the clock during the seven days in which 90 rounds were fired with an average pull of 6 ft. 4 in., using 6,300 lb. of polar ammon gelignite. The best time for a complete cycle of operations was one hour 30 minutes. Equipment used on the operation comprised an "Eimco 21" rocker shovel and Holman SL3 drills using Atlas Diesel Coromant steels. The spoil was removed in Hudson 40 cu. ft. mine cars shunted by B.E.V. battery locomotives and hauled by Ruston & Hornsby diesel locomotives. Each member of the tunnelling crew has received a watch in recognition of this achievement. The Consulting Engineers are Sir Murdoch McDonald & Partners.

European Heavy Power Equipment

THE annual review for the year ended April 1, 1955, of orders and deliveries of heavy power equipment has just been issued by the O.E.E.C. It is, of course, concerned only with European manufacturers, and in the case of water turbines and alternators includes only units of 10 MW and over.

In this field there was a recession in deliveries during the twelvemonth under review, but the orders in hand at the end of the period gave promise of a record output for the succeeding year. Deliveries of water turbines were substantially lower than expected, and only 3,935 MW left manufacturers' plants, arrears totalling 607 MW. On the other hand, new orders exceeded the rate of shipments by 1,602 MW, increasing the total of orders from 10,071 MW to 11,673 MW, 5,537 MW of which represents new orders.

Shipments of waterwheel alternators amounted to only 3,437 MW and were even below 1952 levels, arrears amounting to 351 MW. For 1955-56, however, a record output of 4,783 MW is forecast. Orders in hand at the end of the period for delivery in 1956 exceed even 1954 consignments. For the first time for the last few years, total order books record a substantial increase to a new total of 11,301 MW. This improvement was made possible by the high rate at which new orders came in, which exceeded the dispatch of generating capacity by 825 MW and totalled 4,262 MW.

In the field of transformers of 5 MVA and over there had been an uninterrupted and phenomenal expansion from 1948 to 1953, but in the twelvemonth under review there was a recession. Total shipments decreased from the 1953 peak of 40,396 MVA to 39,096 MVA and deliveries exceeded new orders by 8,786 MVA. The transformer figures, of course, cover units for service in thermal as well as hydraulic systems.

Egypt's High Dam Project

WITH the signing of the contract between the Egyptian Government and Sir Alexander Gibb & Partners, the London consulting engineers, the High Dam development, which promises so much for the economic well-being of both Egypt and the Sudan, detailed plans can be formulated. Model tests conducted at the Grenoble laboratories of the Sogreah establishment have already confirmed the broad outlines of the preliminary plans, and it is understood that the International Bank for Reconstruction and Development have agreed to support the scheme.

The dam will have a maximum height of 330 ft. above river-bed level and is estimated to store 130,000 million cu. m. of water, that is, about 26 times the capacity of the present Aswan dam. The cost is estimated at between £210 and £240 million. In addition to adding two million acres to the cultivable acreage of Egypt the scheme would also produce electrical energy to the tune of 4,250 million kWh per annum, equivalent to ten times Egypt's present rate of consumption. The storage lake created by the dam will be 400 miles long and will extend 150 miles into Sudanese territory. The Sudan is thus vitally concerned but will also reap immense benefits, including the doubling of her present supply of irrigation water. When the scheme is completed in about ten years' time it is estimated that virtually no Nile water will flow into the Mediterranean except in times of exceptional flood. At the present time from 30 to 40 per cent. escapes into the sea.

Addition to Tummel Valley Scheme

THE Secretary of State for Scotland has made an Order for additions to the Tummel Valley scheme to include two new generating stations with a total capacity of 3,900 kW and an annual production of 18,300,000 kWh. One of these will be erected on the eastern shore of Loch Ericht at the outlet of the existing tunnel from Loch Garry and will make use of the difference in level between these two lochs to produce approximately 9,000,000 units of electricity annually from an installed capacity of 1,500 kW. The second station will be built on the Allt Cuaich in Glen Truim to use the difference in level between Loch Cuaich and the upstream end of the existing aqueduct which

leads water from the Allt Cuaich to Loch Ericht. It will have an installed capacity of 2,400 kW and will produce about 9,300,000 kWh of electricity annually.

The scheme also includes the diversion of several streams to the Board's reservoirs, the estimated annual output from these diversions being 10,000,000 kWh. The Order also provides for the enlargement of the existing aqueduct from the Dunalastair reservoir to the Tummel Bridge generating station, so that the level of water in the aqueduct can be raised and the kilowatt capacity and output of the generating station increased. The outflow from this generating station and from the Errochty generating station flows into Loch Tummel and this additional water will be used again farther down the Tummel Valley to increase the output of the generating stations at Clunie and Pitlochry.

Early Start on Shire Valley Contemplated

THE Rhodesian Government has accepted a motion that provision should be made for an early start on the first stage of the Shire Valley project. In the report of the consulting engineers, Sir William Halcrow and Partners, which was summarised in our September issue, it was noted that Lake Nyasa is subject to a cycle of variations in mean annual level extending over a long period of years. This cycle is now in its downward phase, and it is feared that unless a temporary dam is built in the near future the development may be impeded for a number of years to come.

It was announced last month that Sir William Halcrow had retired from the partnership of Sir William Halcrow & Partners, although he would continue to be associated with the firm as a consultant. This well-known firm has acted as consulting engineers for important hydro-electric schemes all over the world.

O.E.E.C. Commission for Energy

THE O.E.E.C. Commission for Energy held its first meetings at the Château de la Muette on September 26 and 27. Sir Harold Hartley is Chairman of the Commission, the other members being Monsieur J. Desrousseaux, Monsieur F. D. Fransen, Professor F. Giordani, Dr. H. Niesz, Monsieur G. A. Tuyl Schuitemaker, Monsieur P. Uri and Dr. F. W. Ziervogel. Mr. Walker Cisler, President of the Detroit Edison Co. and Chief Power Consultant of the American International Co-operation Administration, is taking part in the Commission's meetings as United States observer.

The mandate given the Commission by O.E.E.C. Council Decision of July 6, 1955, was largely inspired by the report drawn up by Monsieur L. Armand, President of French Railways, acting as Consultant to the Organisation. Following this mandate the Commission concentrated on problems requiring definition and questions to be put to Governments of Member countries and other interested circles with the object of obtaining the necessary information to enable it to forecast the needs and the output of energy in Europe in 1960 and 1975. These questions include problems of price, investment, manpower and international exchanges of the various forms of energy, together with technical problems of their production, transmission and utilisation. In addition, the Commission has decided to establish contacts with certain Governmental and private international organisations, together with specialised professional bodies. The Commission will thus be able to obtain a general view of the situation regarding energy in O.E.E.C. Member countries, which will constitute a basis for the suggestions or recommendations which the Commission is required to formulate.

New Swiss Ventures

THREE new hydro-power companies have been recently registered in Switzerland. The Simmentaler Kraftwerke A.G., in Erlenbach, has been promoted for the purpose of developing the Simmental streams and Stocken lakes in three stages. Work is already in progress on the first stage, which consists of the diversion of two torrents into a compensation reservoir from which a penstock will feed the Erlenbach plant; the two subsequent stages are to utilise the storage provided by the Stocken lakes, as well as the power resources of the Simme. The development, once completed, is planned to produce a yearly average of 50 to 55 million kWh. The Kraftwerk Sarneraa A.G., in Alpnach, contemplates the development of the Sarnen-Alpnach stretch of the Sarneraa river, with a power station at Alpnach on which preliminary work has started. The Kraftwerke Engelbergeraa A.G. intends to carry out the development of the unexploited sections of the Engelbergeraa valley.

French Progress

IN 1954 the consumption of electricity in France at 45,300 million kWh was 9 per cent. above that of 1953, and 35 per cent. as compared with 1950. A plea for the acceleration and enlargement of the programme set up under the second Monnet plan, as well as for the elaboration of a third (seeing that demand seems to be increasing annually at a rate of 8 to 9 per cent.) is made by Electricité de France. The second Monnet plan has a target figure of 70,000 million kWh in 1960-61. The report refers to the project for obtaining power from the tides on the Brittany coast, the prospect for which is improved by the development of the reversible turbine which will function when the water level is rising, as well as when it is failing.

North-west Territories Power Commission

THE Government of Canada has issued a report of the Northwest Territories Power Commission for the year ending March 31, 1955. This gives brief particulars of the power plants in operation, their output, together with a statement on the financial position. The Commission was created by an Act of Parliament in 1948 and is empowered to construct and operate electric power plants in the Northwest and Yukon Territories, on a commercial basis. Over the past seven years its operations have resulted in the establishment of an 8,350 h.p. hydro-electric development on the Snare River, a 3,000 h.p. hydro-electric plant on the Mayo River, and a 520 h.p. diesel plant to supply Fort Smith. The power is sold on a commercial basis chiefly to mining companies who contribute over 80 per cent. of the revenue, but the current supplied to domestic and other small-scale users is of great value in providing amenities to the rather isolated communities who inhabit this remote area.

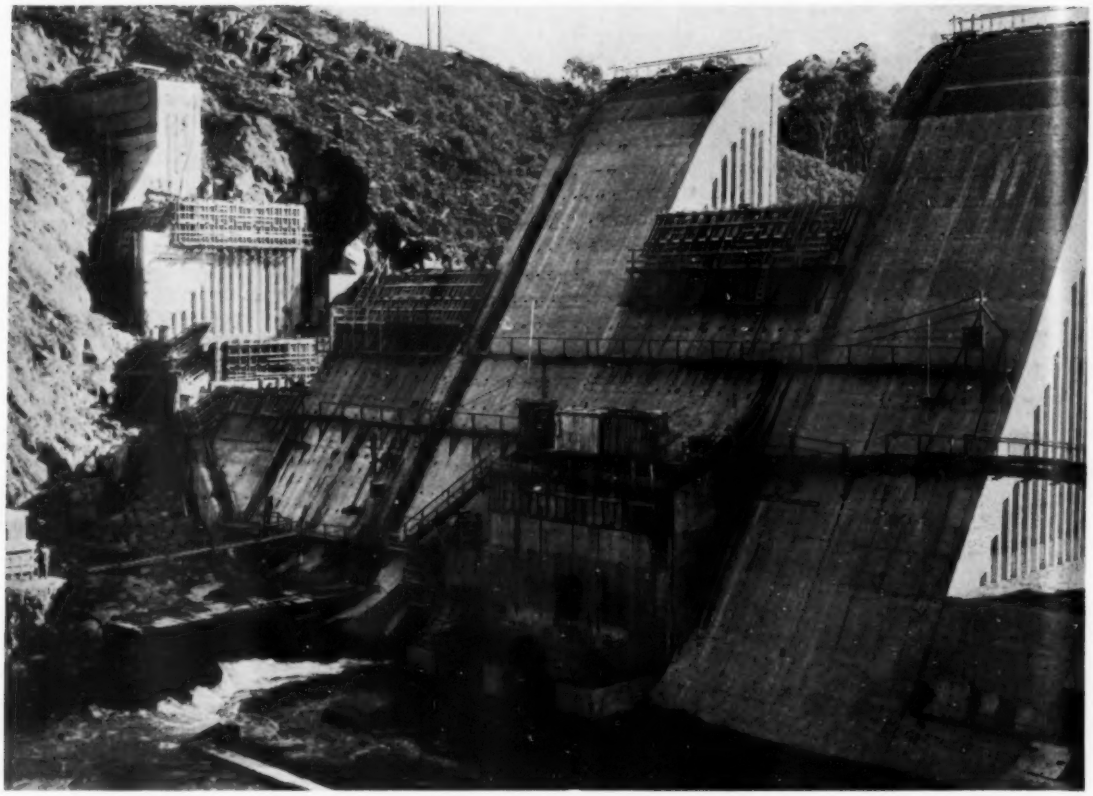

Fig. 1. Downstream view of Pine Tier dam during construction

Pine Tier Dam

We reproduce the substance of a paper* presented to the Tasmania Division of the Institution of Engineers, Australia, by L. S. Whitham, M.E., who is a member of the design staff of the Hydro-electric Commission of Tasmania

THE Tungatinah power development is the third major scheme undertaken by the Hydro-Electric Commission to develop the power potential of the upper reaches of the Derwent Catchment. The map, Fig. 3, shows the geographical relation between the two existing developments, Great Lake and Tarraleah, and the Tungatinah scheme.†

This first stage entails the diversion of the Nive River and its tributary, the Clarence, into a series of storages—Bronte Lagoon, Brady's Lake, Lake Binney and Tungatinah Lagoon—on an elevated plateau on the east bank of the river. These storages were created by building earth and rock banks across the outlets from four marshes. The water will be conveyed from Tungatinah Lagoon (top water level S.L.2135)‡ by unlined tunnel and steel penstocks to the Tungatinah station on the left bank of the Nive a quarter of a mile upstream from Tarraleah power station. The power house will ultimately contain five alternators driven by vertical-shaft Francis turbines each rated at 35,000 h.p. under a static head of 1,005 ft.

The second stage is centred upon Lake Echo where a dam will provide a storage of 600 sq. mile ft. above the outlet conduits with top water level at S.L.2775. Water from the Little Pine River (a tributary of the Nive) will be diverted into the Ouse River, and the combined flow into Lake Echo. The regulated outflow from the lake will be diverted from the Dee River through a tunnel into Brady's Lake and thence to Tungatinah Power Station. The fall of 568 ft. be-

* This paper appears in the *Journal of the Institution of Engineers, Australia*, Vol. 27, No. 4-5, p. 121.
† See WATER POWER, December, 1954, p. 471.
‡ S.L.=Standard Level in feet above mean sea level.

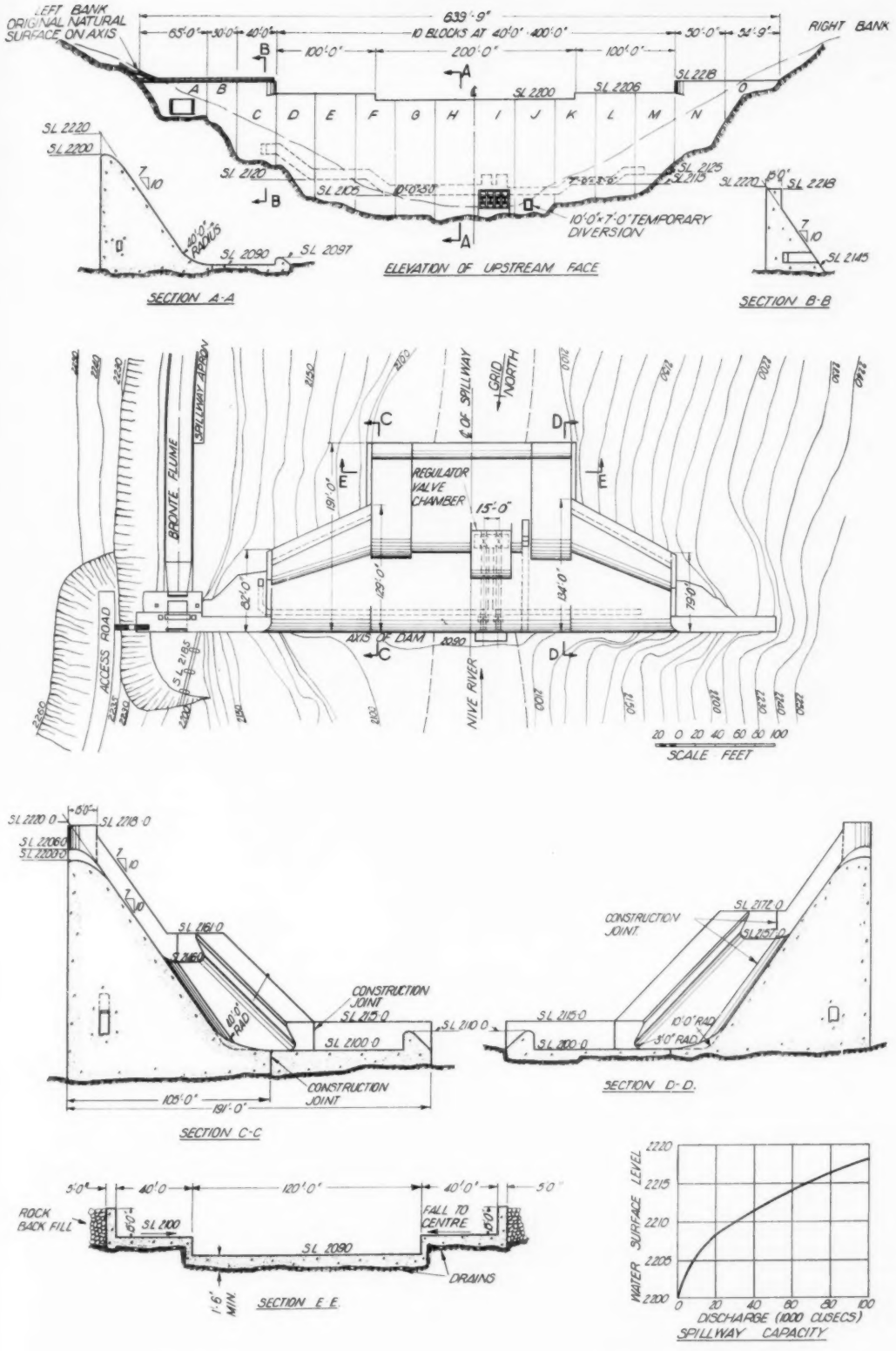

Fig. 2. Upstream elevation, plan and sections of Pine Tier dam

tween Lake Echo and the Dee diversion will be utilised to drive one 42,000 h.p. turbo-generator.

Nive River Diversion

The first scheme for the diversion of the Nive River was to build a dam in the gorge adjacent to Bronte Park town site. Several dam sites were investigated in conjunction with a range of dam heights and a variety of methods of diverting the water to Brady's Lake. With the dam in this location it would have been possible to provide considerable storage in the Nive Valley.

When detailed geological investigations were completed, however, it was found that the foundations for a concrete dam were not as good as expected and that the volumes of excavation and dam were far in excess of the original estimates. It was doubted whether a high "fill" dam could be built in the wet climate of Bronte with the materials available in the Central Highlands.

Attention was then directed to an alternative layout involving a lower concrete dam in Pine Tier Gorge three miles upstream from the Bronte Gorge and a surface conduit some six and a half miles long to carry the water to Bronte Lagoon and thence into the pipehead lagoons. It was uneconomical to provide any storage in the Nive Valley in this proposal.

Studies of costs, availability of materials and construction schedules favoured the second scheme, which was consequently adopted.

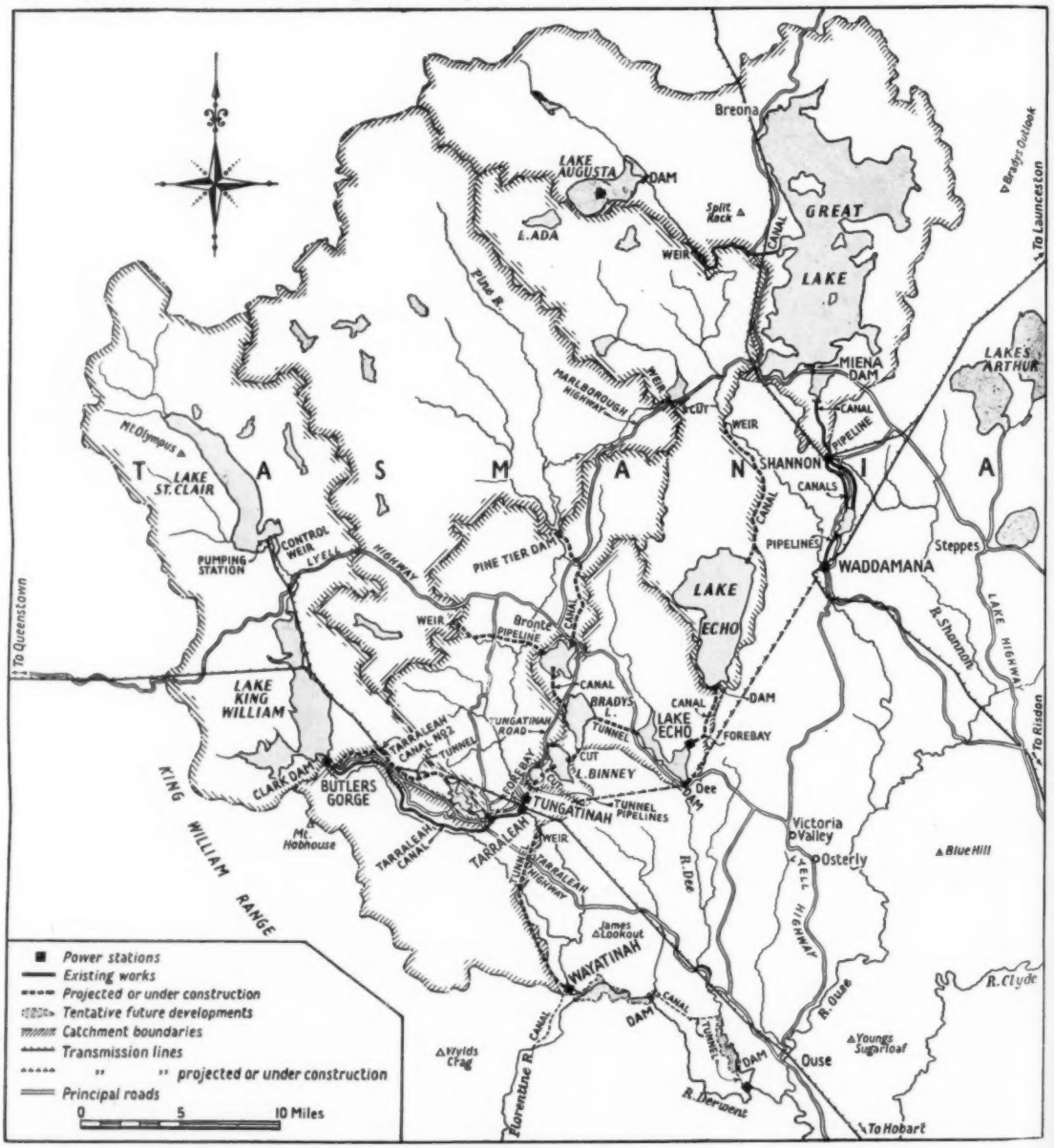

Fig. 3. Map of the River Derwent development including the proposed Lake Echo and Wayatinah schemes

Pine Tier Dam

Pine Tier dam is a mass-concrete gravity structure approximately 640 ft. long at the abutment crest level (S.L.2218), with a central overflow section 400 ft. in length. The general arrangement is given in Fig. 2. There are two scour outlets at S.L.2100 in the river bed and an offtake structure for Bronte flume in the left abutment. The maximum height of 121 ft. occurs in the central spillway where the crest is at S.L.2200. Approximately 105,000 cu. yards (solid measure) of clay, boulders and rock were excavated in the foundations for the dam and spillway structures which contain 90,000 and 10,000 cu. yards of concrete, respectively. The estimated cost of the dam is £A1,456,000. Fig. 1 shows the dam under construction, and Fig. 4 is a view of the completed dam.

Geology of Pine Tier Dam Site

Below the junction of the Nive and Pine Rivers the combined stream flows in a gorge in dolerite for a distance of 3,000 ft. This gorge has been cut down along a dominant system of north-south joints dipping at about 80° to the east. There are no major faults in the gorge. Diamond drill holes were put down in the narrowest section of the valley to determine the best location for the dam.

Excavation revealed that the minor joints are irregular and very close, the spacing varying from less than an inch up to a few feet. The dolerite is broken and weathered to a considerable depth, and it is estimated that the average cover to rock along the sides of the gorge is 25 ft. In Blocks D, L and M (see Fig. 3) excavation was carried to a general depth of 50 ft. (vertically) to expose sound rock, the deepest pocket being 55 ft. below the original surface. The seams exposed down to this depth were tightly packed with clay, with no evidence of calcite. The rock in the river bed was generally very sound, only one minor fault requiring special attention, but elsewhere the depth to rock was usually greater than expected, the discrepancies between sound rock levels as estimated by interpretation of the 1 in. cores and as actually exposed being frequently as large as 20 ft.

Selection of Type of Dam

In selecting the type of dam no preliminary designs of various types were undertaken, but the following points were considered in choosing a mass-concrete gravity dam:

(1) The magnitude of the probable floods and the narrowness of the valley made an overflow type of dam the obvious choice.

(2) The rapid construction schedule envisaged required a simple river diversion, including overtopping of the dam during construction.

(3) Preliminary investigations showed that there was considerable doubt as to whether the abutments were strong enough for an arch dam. As the design of an arch dam involves the selection of a section and its analysis, i.e., a trial-and-error process, this meant that no serious work on an arch could have been begun until the abutment profile had been determined with some certainty. As there was a distinct possibility that the rock would be found unsatisfactory the delay in reverting to designs for a gravity dam would have made the scheme untenable.

(4) The large differential block heights demanded by a rapid construction programme involving overtopping during construction are not permissible in an arch dam where the strength depends upon the successful grouting of the contraction joints. A "high" block with one or both sides exposed for a long time will have almost completed contracting before the adjacent "low" blocks are placed, reducing the width of the contraction joint gap below the minimum required for grouting (about 0·030 in.).

(5) The shortage of carpenters for making and setting of formwork was of prime importance.

(6) Buttress dams were discarded because of design time, formwork difficulties and the requirements of a new construction technique compared with that for mass concrete as well established within the Commission.

(7) A gravity dam would permit overtopping during construction, unlimited differential height, random block sizes and had the advantages of simplicity of formwork and construction and a minimum of embedded fittings such as grout piping, stops, outlets and risers, cooling water pipes and reinforcing steel.

(8) A mass concrete dam would show a large saving, not easily calculable, because the equipment, personnel and technique for mass concreting at Clark dam were available for Pine Tier dam[1]. The importance of these factors may be gauged by the fact that the cost of concrete at the mixer house rose from £2 17s. 3d. per cu. yard at Clark Dam (1946-9) to only £4 12s. 0d. per cu. yard at Pine Tier Dam (June, 1952), an increase of 61 per cent., despite a 66 per cent. increase in the price of cement delivered to the site and a 102 per cent. increase in the average basic wage over the periods quoted.

Estimate of Maximum Flood

For dams designed previously statistical methods had been applied to flood prediction problems. However, as no suitable period of records existed, and the fallacy of forecasting maximum floods from the frequency of past floods has been exposed, a new approach to the problem was sought[2].

It was found that when unusual flood discharges in Tasmanian rivers were plotted, Creager's curve with $C = 30$ formed a limiting curve[3]. To estimate maximum possible floods in the future the flood indicated by the $C = 30$ curve was taken and an allowance made for possible greater floods in the future. The catchment area above the dam is about 300 square miles, giving a maximum possible flood on this basis of 100,000 cusecs.

An attempt was made to check the estimate by using synthetic unit hydrograph methods. In the first instance maximum possible rainfall figures for a thunderstorm type storm were assumed, based on some Victorian figures available, and the maximum possible flood indicated was 100,000 cusecs. Later, estimates of the maximum possible rainfall from a thunderstorm in Tasmania were published by Mr. J. Walpole, of the Commonwealth Weather Bureau, and the maximum possible flood then indicated was 70,000 cusecs.

However, the constants used in the synthetic unit hydrograph calculations were assumed without any check being possible. Furthermore, it is possible that the flood as indicated by the thunderstorm could be exceeded. It was therefore decided to retain the original estimate of 100,000 cusecs. The scores of lagoons and lakes in the upper reaches of the catchment undoubtedly have a great effect in reducing flood peaks. The largest flood known in the Nive at the dam site is 21,000 cusecs in June 1952.

Preliminary Design of Spillway

Until the length and elevation of the spillway crest of an overflow dam are determined, no firm dimen-

sions for the dam cross sections for either the spillway or abutment can be computed. Due to the delays occasioned by changing from the Bronte to the Pine Tier dam site, the time available for comparative designs and model tests was very short. As excavation had not proceeded far enough to give positive indications of the final foundations, the spillway design was based on estimated rock levels. To meet the fast construction schedule required by the Commission, concrete placing was well advanced in some blocks long before excavation was started in other portions of the dam. As a result it is now apparent that the spillway design could have been simplified on the left bank, although to have awaited final excavation results before completing the design would have delayed construction by over a year.

Preliminary computations showed that the most economical length of spillway crest would be 400 ft. As the river channel at normal flows is about 120 ft. wide and at maximum flood flow 200 ft. wide, some means had to be devised for returning the water from the outer 100 ft. of spillway crest on each side to the central river bed. The weir was stepped so that floods of 10,000 cusecs and less are confined to the central 200 ft. of the spillway. The bucket radius of 40 ft. for this section of the spillway was chosen after examination of drawings of similar overflow dams. It was possible to achieve economy in excavation and concrete by stepping the elevations of the buckets and aprons to follow the rock profile.

Spillway Model

The only way that tests could be made without undue expense and loss of time was to utilise to the best advantage the existing facilities in the hydraulic laboratory of the University of Tasmania. The flume available necessitated a scale of 1 : 120, and the existing pumping layout limited the tests to a prototype flow of 83,000 cusecs. Although this scale was very small, the close similarity in behaviour of the 1 : 100, 1 : 60 and 1 : 20 scale models of the Hoover dam spillways led to the belief that the tests performed gave a good picture of how the prototype would behave[1]. As one million horsepower are to be dissipated when the spillway discharges at full capacity emphasis was placed in the model tests on the efficient destruction of this energy.

The initial series of tests was to determine the best shape of the walls of the side channels. The first design was based on overflow dams in the Galloway power scheme, Scotland and the Mangahao hydro-electric scheme in New Zealand, with plane floor slabs and vertical walls. A test run showed these to be totally inadequate for the large flows obtaining at Pine Tier dam, as the jet rose vertically against the wall to the level of the crest of the dam. After extensive trials the curved deflectors (Fig. 3) were evolved to throw the jet back into the channel. The wall height and slope and the radius of the fillet were varied until a shape satisfactory from both hydraulic and structural viewpoints was developed. The water followed an approximately helical path down the channel, turning about two-thirds of a revolution in its passage from top to bottom of the channel. This action is demonstrated in Fig. 5 which is a vertical photograph of the model showing the right bank deflector with a total discharge over the dam of 80,000 cusecs. The side channel walls were oriented to ensure that the jets met in the centre to effect the mutual destruction of their energy and that the downstream component of velocity was as small as possible. The velocity of the central nappe was thus considerably reduced in the stilling pools by sharing its momentum with the water discharged from the side channels. The disparity of the bank slopes and the off-centre location of the valve house were responsible for the unsymmetrical layout of the side channels.

The length of the apron was chosen arbitrarily and only a few experiments were made to find the effect of variations, as the length of the model was insufficient to reproduce downstream river conditions with accuracy. The same degree of uncertainty applied to the height of the apron weirs.

Final Design of Spillway

It was directed that to reduce the volume of concrete in the overflow section the crest curves should be designed to fit the nappe of a 60,000 cusecs discharge, provided the dam was adequately stable with an overflow of 100,000 cusecs. The design heads for the central and side crests were then chosen so that both curves would be tangent to the one plane on the downstream face. The central notch, 200 ft. long with crest at S.L.2200, has a nominal design head of 13·2 ft., and the side weirs, each 100 ft. long with crests at S.L.2206, have a nominal design head of 9·3 ft. The layout is shown in Figs. 2 and 4.

Design of the spillway was greatly simplified by the absence of crest gates. As the valley is uninhabited, the rise in level during a flood is unimportant. The valley storage in Pine Tier Lake above spillway crest level is insignificant—namely, 7 sq. mile ft. between S.L.2200 and S.L.2218—and has negligible effect in reducing flood peaks.

As there was considerable overbreak behind the channel deflector walls, no reliance was placed on backfill assisting the walls to resist the overturning force of the spillway nappe. The mass-concrete deflector walls were designed to withstand the dynamic thrust of the jet without developing tension. The overhanging portion was reinforced to prevent the development of tension or shrinkage cracks.

As the foundations for the apron slab were not as good as had been expected, the design requirement of 18 in. thickness was increased to 5 ft. or to solid rock, whichever was the less. The slab is not reinforced, and has construction joints at 20 ft. centres in both directions.

Had it been possible to forecast accurately the depth of excavation for Blocks D, E and K and the extent of the overbreak downstream from them, there would have been no need to resort to the side collection channels. The S.L.2200 crest and plain bucket could have been extended to a length of approximately 300 ft. The lake level for a discharge of 100,000 cusecs would then be S.L.2218·5.

Stress Analyses

Data used in the design of the dam are given below:

1. Weight of concrete 160 lb. per cu. ft. This was based on tests at Clark dam where densities as high as 167 lb. per cu. ft. were obtained. Tests at Pine Tier dam show that the density averages 160 lb. per cu. ft. with a range of 158-163 lb. per cu. ft.
2. Maximum upstream water level S.L.2218.
3. Maximum tailwater level S.L.2120.
4. Uplift was assumed to vary from full water pressure at the upstream face to zero or tailwater pressure at

Fig. 4. The completed Pine Tier dam, with the Bronte flume in the foreground, and showing the design of spillway, side channels and stilling basin

Fig. 5. Vertical photograph of spillway model showing the action of the deflector wall

the downstream face, these pressures to be applied to two-thirds of the horizontal area of the base and to two-thirds of the horizontal areas of the concrete sections at elevations investigated above the base.

5. Assumed ultimate shearing strength of concrete and rock 400 lb. per sq. in.
6. Assumed coefficient of friction between rock and concrete, and between two concrete surfaces, 0·75.

Vertical and inclined stresses at the faces of the dam were computed by conventional methods. Distribution of horizontal shear stress was computed by Leliavsky's method[5]. Shear-friction safety factors were calculated as:

$$\frac{(\text{Net vertical load} \times \text{coeff. of friction}) + (\text{plan area} \times \text{shear strength})}{\text{Net horizontal load}}$$

The maximum vertical compressive stress at the upstream face of 151 lb. per sq. in. occurs at the base of Block H at S.L.2079 for dead load only. The largest tension of 2 lb. per sq. in. occurs in the same block at S.L.2099 under maximum flood conditions.

The maximum stress parallel to the downstream face of 153 lb. per sq. in. occurs at S.L.2079 in Block H under maximum flood conditions. With the pond empty a small tension of 2 lb. per sq. in. occurs at the base of the maximum abutment section—S.L.2134 in Block C. The minimum shear-friction factor of safety under flood conditions is 11·7 at the base of the deepest blocks in the central and side overflow sections. The minimum for the abutment section under the same conditions is 16·9.

The planes of the joints between the dam and the deflector on both banks had been determined from the assumed rock levels and fixed by advancing construction before it became obvious that in some sections the sound rock was considerably deeper than expected. The vertical joint face between dam and deflector reached a maximum height of 30 ft., posing two important problems, (*a*) the stability of the dam blocks with a smaller base-height ratio than designed and (*b*) the stability of the deflector blocks with increased height. These problems are similar to those in wide dams where the longitudinal slices must be made to act monolithically. The traditional solution of grouting the joints could not be employed as the

dam had virtually completed cooling and contracting before the deflector walls were placed and the joint would not open to a groutable width. The solution adopted was to take care to obtain a good bond on the vertical joint, and complete concreting the deflector blocks before filling the lake so that plastic deformation could close the joint and permit the transfer of shear stress across the interface.

Tests on the concrete placed up to January, 1953, showed that the average 28-day strengths of the Class C and Class D (see below) concretes were 3,300 and 2,100 lb. per sq. in., respectively. With the modified cement in use there is a considerable increase in strength of 5,400 lb. per sq. in. for Class C and 4,100 lb. per sq. in. for Class D concrete.

Gallery

For purposes of foundation grouting and drainage and for access to the low-level outlet works a gallery has been provided from Block C to Block M, 17 ft. from the upstream face and averaging 20 ft. above the foundation rock (see Fig. 2). A short branch in Block I extends to the regulator valve chamber. The galleries are 5 ft. wide throughout, 10 ft. high from the entrance in Block C to the gate valve chambers and 7 ft. high in the remainder.

The height from the entrance to the valve chamber was increased to permit the parts of the gate valves to be transported through the gallery, as it was expected that concrete placing would be completed almost a year before the valves arrived. However, by a considerable improvement on the promised delivery the gate valves arrived in time for the parts to be hauled through the transverse gallery in Block I, and a 7 ft. high gallery throughout would have sufficed. Hooks are set in the ceiling over the flights of stairs for handling parts of the valves and the drilling and grouting equipment. A 12 in. diameter ventilation pipe extends from the end of the gallery in Block M to the downstream face of Block N to prevent foul air accumulating in the gallery.

Treatment of Foundations

The final rock surface was cleaned by air and water jets and all loose material scrupulously removed just prior to concrete placing.

For blanket or consolidation grouting 1¼ in. diameter holes were diamond-drilled on a rectangular grid at 10 ft. centres to a depth of 20 ft. All holes were pressure-tested with water. This showed that many holes were loose and interconnected by clay-filled joints. Many joints were cleaned out in this way, and washing was continued until water from adjacent holes showed no sign of discolouration.

The blanket grouting was carried out after a minimum of 10 ft. of concrete had been placed over the foundations. A pressure of 25 lb. per sq. in. was used generally, but 30 lb. per sq. in. was used in places on tight holes. Where the water tests had revealed excessive leakage, extra holes were drilled and grouted after the grouting of the ordinary holes.

Typical results of the grouting programme are shown in Table I.

TABLE I. TYPICAL RESULTS OF BLANKET GROUTING

	H Block	I Block
Total quantity of cement injected	43,428 lb.	14,750 lb.
Number of holes	45	39
Average quantity of cement per hole	965 lb.	374 lb.
Maximum in one hole	4,935 lb.	2,256 lb.
Total quantity of grout	1,490 cu. ft.	650 cu. ft.
Total length of holes drilled	1,000 ft.	780 ft.
Cubic feet of grout per foot of hole	1·49	0·83

Curtain grouting under the upstream toe will be effected through holes drilled from the gallery or the downstream face of the dam after all the concrete is placed and before the lake is filled. The holes will be drilled to a depth of not less than 60 ft. and grouted progressively downwards. The arrangement of the grouting and drainage holes is shown in Fig. 6. On completion of the grouting, drainholes will be drilled from the gallery into the rock 5 ft. below the blanket grouting. An indication of the effectiveness of this type of grouting in dolerite country in preventing leakage is given by the figures for Clark dam. On December 14, 1951, when the maximum depth of water was 166 ft. the total leakage into the gallery was only one-fortieth of a cusec of which 40 per cent. was coming from one drainhole.

Provision has been made for measuring the water pressures which would exist in the foundations if the drains were clogged.

To take the measurements the drain pipes will be plugged at the outlet into the gallery drain, and a pressure gauge connected to the tee in the pipe. From these measurements the effect of the curtain grouting in reducing the uplift intensity can be assessed.

The upstream foundation key, a feature of gravity dams of a few decades ago, was eliminated on the grounds that the successful development of foundation grouting has obviated the need for an increased leakage path, and any structural value it may have might be offset by the damage to adjacent rock during blasting.

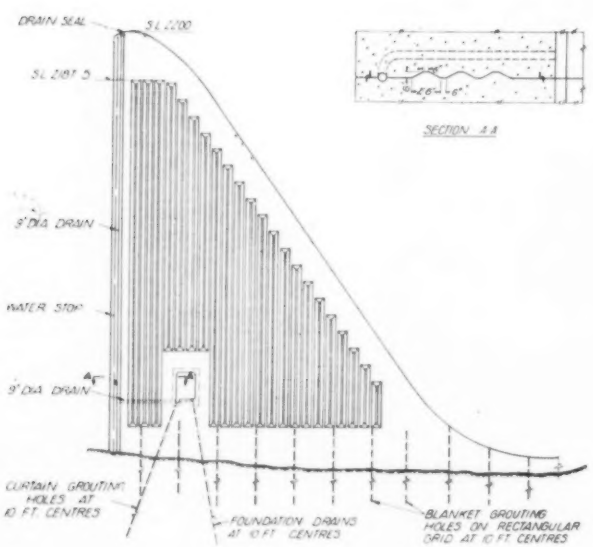

Fig. 6. Elevation and detail of a typical contraction joint

Contraction Joints

The dam has been built on the conventional block system with the block lengths (30-65 ft.) chosen mainly to suit construction methods. The value of keys on the joint faces in a gravity dam is problematic, and the fact that they entailed the employment of carpenters in every stage of fabrication and installation was the main argument in favour of their elimination at Pine Tier dam. However, as their overall cost is only a very small item in the total cost of the dam, it was decided to key the joints as an additional factor of safety against sliding. A few large keys 12 ft. wide by 1 ft. deep were specified initially with the intention of keeping carpentry work to a minimum. When it was discovered that these would prevent the use of the standard cantilever forms as used at Clark dam they were replaced by the conventional 18 in. by 6 in. keys. A 32 oz. copper waterstop projecting 5 in. into each block is provided across each joint 1 ft. from the upstream face, with "wings" 2 ft. long at every horizontal construction joint. Waterstops are also fitted round the gallery at each contraction joint. A 9 in. diameter drain hole 1 ft. behind the waterstop leads any leakage either to the gallery or to the downstream face. Removable concrete covers are provided on the crest so that the holes may be cleaned out if necessary. The joints were not grouted. Fig. 6 is an elevation of a typical joint.

Internal Drainage

A 12 in. wide drain runs the full length of the main gallery collecting leakage from the joint and foundation drains into a sump in the gate valve chamber. From these the leakage is piped to the regulator valve chamber and released on to the spillway apron through a non-return valve. Two 12 in. diameter pipes through the central apron weir at S.L.2090 drain the apron.

Crack Prevention Programme

The heat of hydration of the cement causes the temperature of concrete to rise for a considerable time after placing. With a fast concreting schedule and concrete containing 445 lb. of cement to the cubic yard, this rise may be as much as 45° F. unless artificial cooling is used. This heat may lead to cracking of the concrete in one of two ways.

If a thick layer of concrete is placed on a "cold" joint (i.e., a rock surface, or a concrete lift from which most of the setting heat has been dissipated) this new concrete hardens in an expanded state due to the heat rise from hydration, and the subsequent contraction on cooling is restrained by the already contracted or rigid base. If conditions are sufficiently unfavourable cracks will extend through the new lift and into succeeding lifts. This type of cracking may be minimised and perhaps prevented by placing the concrete on cold joints in a number of shallow lifts from which the heat is lost before hardening is complete.

Cracking may also result if a number of consecutive lifts are placed too rapidly. The interior of the lower lifts in which heat generation is still continuing will be heated by the overlying lifts and tend to expand. The cooler faces which have hardened more rapidly than the interior may crack if they are unable to elongate at the same rate as the interior.

Concrete made with Tasmanian dolerite as both fine and coarse aggregate has the extremely low heat diffusivity of 0·022 sq. ft. per hour, so that it is of prime importance to keep the maximum temperatures as low as possible.

A seven-point programme for preventing or minimising the formation of cracks was set out and carefully adhered to in the field:

(1) A modified cement was used in the dam. Specification requirements for heat of hydration and specific surface were—"At age 7 days the heat of hydration shall not exceed 70 calories per gram. At age 28 days the heat of hydration shall not exceed 80 calories per gram. The specific surface of the cement as measured by the Pecover-Banks air-permeability apparatus shall be not less than 3,300 sq. cm. per gram." Average test results up to July, 1952, were 61·7 calories per gram. (range 59-64), 74 calories per gram (range 72-76) and 3,346 sq. cm. per gram, respectively.

(2) A low cement content was used. In the first half of the concrete placed the cement content was 418 lb. of cement per cubic yard of concrete (Class C). Later, Class D (348 lb. per cu. yard) was used as a hearting between faces of Class C concrete.

(3) On foundation rock which was substantially level the concrete was placed in half-lifts (2 ft. 6 in.) at a minimum interval of five days until at least 80 per cent. of the foundation rock was covered before normal 5 ft. lifts were started. Where the side slopes were steep (Blocks B, D, M, N, where the rises were 32, 35, 33 and 42 ft. in block widths of 30, 40, 40 and 50 ft., respectively, and Block A where the rock rises almost vertically) the concrete was placed in 2 ft. 6 in. lifts at minimum intervals of five days until the plan area of rock to be covered by a 5 ft. lift was not greater than 20 per cent. of the area of top of the preceding lift.

(4) On "cold" joints, i.e., where a lift surface was exposed for 14 or more days the procedure for placing on rock was followed.

(5) Minimum time permitted between lifts or half lifts was five days.

(6) Galleries and conduits were sealed for as long as possible to prevent rapid local cooling.

(7) Reinforcement was placed over the conduits, galleries and diversion outlet where shown desirable by the stress analysis.

Designs were prepared for artificial cooling and a small quantity of fittings obtained for use in any block where construction exigencies might require a faster schedule than in (5) above. Actually no artificial cooling was required.

Bronte Flume Intake

The intake works for the Bronte flume are situated in Block A on the left bank. They comprise an approach channel excavated to S.L.2185, a trashrack structure upstream from the dam and an automatic control gate housed in the abutment block. Economic considerations fixed the flume capacity at 1,650 cusecs. The duration curve for the Nive River at Pine Tier dam for the "critical" design period shows that the natural flow in the river is equal to or less than this figure for 93 per cent. of the time. When the Little Pine River is diverted into Lake Echo the mean flow at Pine Tier dam will drop from 590 to 490 cusecs, and the flume will divert the full river flow for 96 per cent. of the time.

The trashracks, consisting of 30 lb. rails at 12 in. centres, extend one foot above the 10,000 cusecs flood level and will be overtopped by floods greater than 16,000 cusecs. Stoplog slots are provided upstream of the gate, but it is not expected that stoplogs will be required because the low-level outlet works can

draw the water level down to S.L. 2185 or below for 90 per cent. of the time.

As the Bronte flume discharges into an unlined earthen canal about halfway between the dam and Bronte Lagoon, close regulation of the flow is important. A long trimming spillway on the flume, which is a common solution in such cases, was impracticable because of the unfavourable terrain. From several types of gates and control devices investigated an automatic hydraulically operated counterbalanced radial gate was selected. Electrical operation was not considered because there will be no permanent power supply after construction is finished. It has been designed to limit the flow in the flume to 1,650 cusecs plus a small operating margin when the river flow exceeds 1,650, and to shut down completely when the discharge of the dam spillway reaches 10,000 cusecs. A short trimming spillway starting 155 ft. from the dam will take care of small excess discharges.

The method of operation may be followed by reference to Figs. 7 and 8. While the river flow is less than or equal to 1,650 cusecs, the radial gate *A* is held in the fully opened position under the action of two counterweights *B*. An 18 in. diameter pipe *C* supplies water from a point in the flume about 200 ft. downstream from the dam to a weir tank *D* which has a triangular notch *E* set approximately six inches below the normal 1,650 cusec level in the flume at the offtake point. The discharge from the notch *E* enters the counterweight shafts *F* and escapes through a nozzle *G*. When the flume flow exceeds 1,650 cusecs the water in the counterweights shafts builds up sufficiently to reduce the effective weight of the counterweights, permitting the gates to partially close. The head-discharge characteristics of the notch and nozzle are so related that for every lake level between S.L.2200·0 and 2205·8 there is only one corresponding stable position of the gate. Over this range in levels the flume flow increases from 1,650 to 1,670 cusecs.

When lake level reaches S.L.2205·8 (i.e., 10,000 cusecs discharge over the dam) sufficient water enters

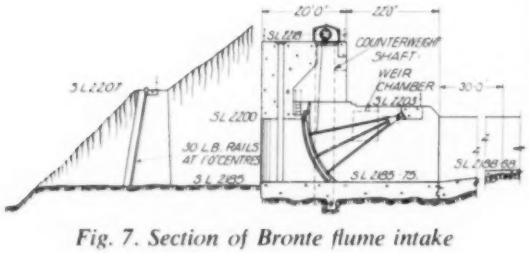

Fig. 7. Section of Bronte flume intake

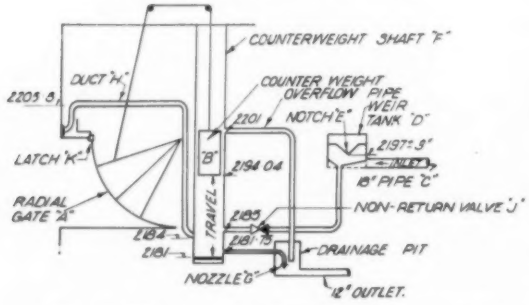

Fig. 8. Diagram of automatic gate control

through duct *H* to flood the counterweight chambers and shut the gate right down. The non-return valve *J* prevents backflow into the weir chamber. Latches *K* prevent the gate from reopening automatically when the flood recedes. The hand hoist must be used for reopening the gates after a flood to limit the rush of water into the empty flume.

The stability of the system was analysed by assuming a small displacement of the gate and computing by a step-by-step process the resulting oscillations. Without friction the period of oscillation is about 60 seconds with very slow damping. However, friction in the side seals, etc., rapidly damps out the motion, rendering the gate quite stable.

The size of the counterweights was adjusted so that they can exert a force exceeding the gate reaction and estimated friction by half a ton to move the gate to the fully open position. To ensure a positive seal when the gate is closed the outflow pipe is at such a level that the effective weight of the counterweights is one ton less than that needed to just close the gate.

A low weir in the flume at the downstream end of the trimming spillway maintains a constant level (for any given flow) at the offtake to the gate mechanism, irrespective of changes in regime in the remainder of the conduit.

Radial Gate

The radial gate, 20 ft. wide by 15 ft. high, was designed to withstand water to S.L.2213·75 (i.e., a flow of approximately 60,000 cusecs) with a working stress of 18,000 lb. per sq. in. The highest stress under maximum flood conditions is about 24,000 lb. per sq. in. Welded connections were used throughout except for fastening the skin plate to its supports. Care was taken in design and construction to minimise distortion due to welding and to avoid crevices where corrosion could start.

The pivots are bolted to a heavy beam spanning the flume, making it possible to support the main cross-members at about the fifth-points with maximum positive and negative moments equal. The maximum load on each pivot with water at S.L.2213·75 is 109 tons. Blockouts were left in the side walls so that the sealing plates of stainless steel could be accurately set. Drop-tight sealing under flood conditions is not required. Rollers at each side of the gate prevent it from binding or scoring the side seals.

Low-Level Outlet Works

A 10 ft. by 7 ft. diversion outlet was left in the bottom of Block J to pass river flows of up to 1,000 cusecs. This was back-filled and grouted when installation of the valves was completed.

There are two permanent 60 in. diameter conduits at 15 ft. centres in Block I a few feet above the original river bed for dewatering the dam for maintenance purposes. The conduits are graded from S.L.2100·00 at the upstream face to S.L. 2099·50 at the downstream face. The entrance to the conduits is protected by a rectangular trashrack structure designed for a differential head of 13 ft. The vertical racks have a clear spacing of 5½ in. and a gross area of 184 sq. ft. per conduit. Since the volume of water retention is so small, no provision has been made for cleaning the racks except by dewatering the dam. The trashrack roof at S.L.2110 could be dewatered for about 60 per cent. of the time. Stoplog slots are provided on the

upstream face of the dam inside the trashrack structure.

Each conduit will discharge 730 cusecs with the water at the spillway crest. The bell-mouthed entrances were designed to prevent cavitation, since the velocity through the pipes at this flow is 37 ft. per sec.[6]. A $\frac{3}{8}$ in. steel lining fitted with labyrinth rings commences four feet from the upstream face to increase the leakage path from the reservoir to the valve chamber.

Hand-operated 60 in. Blakeborough gate valves are fitted as service valves at the upstream end of each conduit. Two 6 in. diameter bypasses are provided so that the gate valves may be opened under balanced pressure. The valves are concreted in up to the underside of the cover flange.

Arrangements had been made for fitting temporary bulkheads over the conduit entrances to enable the gate valves to be erected in the dry, but as these valves, were installed before the river was diverted into the flume, the bulkheads were not necessary.

Bopp and Reuther 1,500/1,350 mm. (4 ft. 11. in/4 ft. 5 in.) manually operated annular slide valves at the downstream end of the conduits are housed in a heavily reinforced chamber projecting from the face of the dam. A hook is provided in the ceiling over each valve as in the gate valve chambers, for a 20 ton hand operated chain block. The valve house will be partially submerged during floods and for this reason wall outlet cones with air admission pipes are bolted to the downstream flanges of the valves. The crest height of the central spillway apron weir was limited to S.L.2097 to prevent the discharge regulators from being drowned by water in the stilling pool. Four 15 in. diameter opening ports are fitted for light and ventilation of the valve chamber.

A few cusecs for the use of riparian owners below Pine Tier dam will be released through a pipe connected to the gate valve bypass piping.

Conclusion

Although much of the material presented in this paper describes conventional practice, the provision of the curved deflector walls in the spillway channels is believed to be a departure from normal practice, and the radial gate control mechanism is an original design.

Pine Tier dam, which forms part of The Hydro-Electric Commission's Tungatinah power development, was designed and constructed by the Civil Engineering Branch of the Commission. Mr. G. T. Colebatch, O.B.E., B.E., M.I.E.Aust., is Chief Civil Engineer, Mr. H. H. Thomas, B.E., M.I.E.Aust., is Deputy Chief Civil Engineer, Mr. J. K. Wilkins, B.Sc., M.I.E.Aust. is Engineer for Civil Designs, and Mr. L. M. Johnson, B.E., is Resident Engineer for the Bronte construction area.

Acknowledgments

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Scottish Hydro Power

In a presidential address to the Association of Supervising Electrical Engineers recently, Mr. Bryan Donkin took as his subject the electrical developments of the North of Scotland Hydro-Electric Board, giving a general survey of their activities in an area which exceeds over 21,000 miles in extent and incorporates a number of islands and other isolated districts. He pointed out that most of the water-power schemes were situated near the west coast where the rainfall was heavier than on the eastern part of the country.

Since it was first constituted in 1943 the Board had invested about £118 million of capital in the construction of hydro-generating plants, transmission lines and local distribution systems. An almost infinite variety of natural conditions were encountered, he added, justifying the use of Pelton wheels for the higher heads, Francis runners for the medium heads, up to 1,000 ft., and Kaplan turbines for the lower heads. From an initial 207 miles of 132 kV transmission lines taken over on vesting day the Board had since erected an additional 953 miles and had at present 200 miles under construction. At the end of 1954 the capacity of plant installed was 594 MW, of which 431 MW (or over 72 per cent.) was in the form of hydro sets, and a further 398 MW of hydro plant was now under construction.

Enfield Cable Contract for Ontario. Enfield Cables Limited, of Brimsdown, Middlesex, England, announce that they have been awarded a contract to the value of approximately $500,000 by the Hydro-Electric Power Commission of Ontario, Canada. The cables will be laid in Ottawa, and they will cross the well-known Rideau Canal in the capital city. Some 2½ miles of 115 kV double-circuit 80 MVA pipeline compression cables (conductor size 300 mcm.) are to be laid by Enfield Cables' Contract Department in Canada. The installation is due to be commissioned on May 1, 1956.

Ansaldo-San Giorgio. A series of catalogues received from this firm, which has its headquarters in Genoa, Italy, give a description (in English) of the large "elastic" transformers, 184,000 kVA, 264,000 V, supplied for the Edison Company's Mese power station; a description (in Italian) of a 12·2 m. (39·6 ft.) diameter umbrella-type 3-phase, 50 cycle, 62·5 r.p.m. generator manufactured for the Porto della Torre scheme; and a description (in English) of the Monte Argento hydro-electric power station and of the plant they supplied there. A final publication, also in English, gives an extensive list of references enumerating the main hydraulic and electrical machinery that Ansaldo-San Giorgio have manufactured at their works.

Effect of Modern Constructional Methods on Tunnel Design

Prof. Dr. techn. L. v. Rabcewicz, Stockholm, shows that modern methods of excavating and lining tunnels driven in unstable rock permit a more economical design of tunnel to be accepted.

PART ONE

WHEN designing the lining of a tunnel in unstable rock it is customary to adopt a type and thickness of lining based on experience gathered from tunnels constructed in similar geological conditions. Sometimes these assumptions are justified by graphical analysis, equilibrium polygons being drawn and stresses calculated.

The fact that a tunnel construction stands for decades without failing does not necessarily prove that its design is economic. Generally, linings have been made a great deal too strong, and still are so made. This is quite understandable, as designers, being aware of the great uncertainty of the problem, try to be on the safe side. Such overdimensioning, though excusable in the old days, must be considered as an economic mistake nowadays, when tunnels are constructed according to modern principles.

Methods of excavating and lining have changed fundamentally during the last few decades, but this has caused the static conditions to change also, as these conditions are not only functions of geological conditions but also depend basically on the way a tunnel is excavated and lined. This important fact is seldom taken sufficiently into consideration when designing a tunnel and preparing the specifications. The following article is intended to examine different methods of excavating and lining a tunnel with regard to the development of load, and to show how advantage can be taken of the improvement in conditions brought about by modern methods to obtain a less expensive design of tunnel.

To simplify the task the geological conditions to be contemplated will be reduced to the most common case of relatively shallow tunnels in unstable, jointed and shattered rock, but without squeezing or swelling effects. Cases of soft material will likewise be excluded as well as problems involving inside water pressure. The range covers the items 1 to 6 in Terzaghi's Table of Rockloads, i.e., from rock described as " hard and intact " up to " completely crushed but chemically intact," with the restriction that the face and the side walls are supposed not to need to be supported for a period of at least one round.[3] Consequently we shall have to deal solely with pressures exerted by the weight of loosened rock, the loosening being caused by the process of excavation and lining.

From the point of view of damage to the surrounding rock the most economic method to drive a tunnel in rock is by milling or chiselling it out by specially built tunnel-driving machines, but this method is practically confined to small sections and long tunnels in homogeneous rock and therefore so seldom used that it can be left out of consideration.

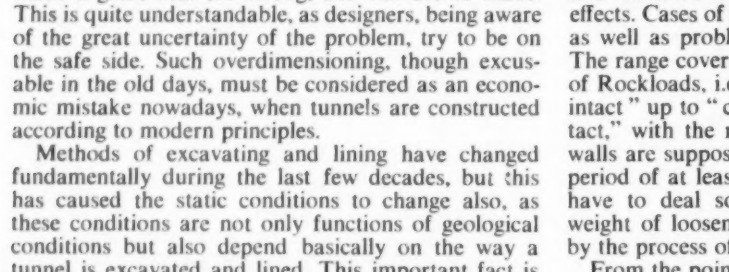

Fig. 1. Smooth roof in solid granite obtained by millisecond blasting (Engineering News-Record)

When excavating a tunnel in the usual way by blasting certain damage to the surrounding rock obviously cannot be avoided. Nevertheless, today's blasting techniques enable the shock effect to be minimised by taking advantage of modern electric delay firing, particularly millisecond ignition. Particular care should be taken when charging the border holes; less powerful charges should be used, the cartridges being set at distances or cut longitudinally and attached to rods in order to reduce the shock as much as possible. In Fig. 1 is shown the excellent result that can be obtained in

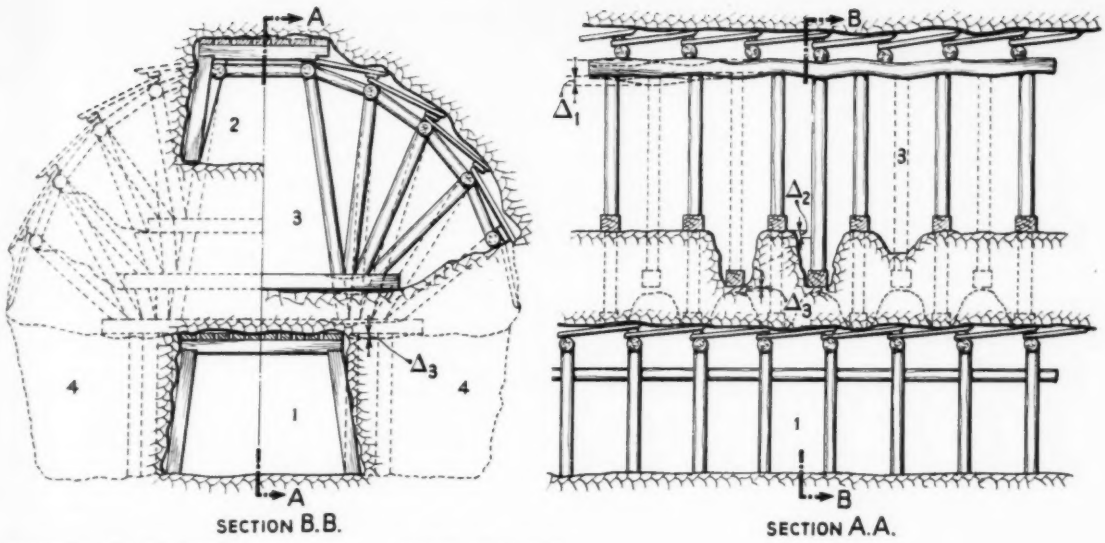

Fig. 2. Tunnel excavation according to the Austrian method showing different stages of excavation and gradual enlargement of the upper heading

this way in solid homogeneous rock, the rock behind remaining perfectly sound.

This is, of course, different in unstable rock. Though the transmission of detonation waves is greatly reduced by joints—either clayfilled or open—the disturbance to the surrounding rock will cause the joints to open and the neighbouring material to move towards the open space. Radial joints will close during this movement to form the natural arch, the span and the radius of which are determined by the geological properties of the rock.

But comparatively speaking, careful blasting is one of the less important factors in minimising loosening of the surrounding rock, the method of driving and the means of temporary support being of much greater importance, especially for the larger sections.

To illustrate this problem more clearly, the excavation of a tunnel in unstable rock will be considered according to three different methods, and these will be compared with regard to the development of load around the tunnel.

From the older tunnelling methods the once most-favoured "Austrian" method, in accordance with which most of the long tunnels in the Alps were built, will be chosen. From the modern systems full-face and heading and benching combined with support by steel arches will be considered, and finally the same methods of excavation combined with roof-bolting.

Austrian method. When excavating a tunnel by the Austrian method (see Fig. 2) a pilot gallery *1* is driven first followed by an upper heading *2*. Then for a length of 6 to 8 m. the upper portion of the section is excavated gradually in steps by placing crown beams longitudinally and bracing them against the centre part of the section, to become supported finally by heavy sills resting on posts, the latter being placed in the somewhat enlarged pilot gallery *3*. Then the abutments *4* are excavated. To understand better the serious amount of loosening inevitably connected with this method, the different stages of excavation are described below more in detail:

When driving the galleries *1* and *2* some minor loosening will occur at the outset due to the general disadvantageous properties of wooden support. Thus, the surfaces of contact between the different members are initially very small and increase to the necessary size by deformation of the material under load. Again, there are deformations caused by bending of the crownbeams, and further, the posts settle in the floor, which is poorly adapted to take load because it has been loosened by blasting and probably by water. When excavating in rocky material, voids between the lagging and the rock face cannot be avoided, and these cause the superposed rock to settle. But the greater part of the loosening occurs when excavating the roof gallery *3*. The enlargement of the upper part between the two galleries proceeds downwards in several steps, as shown in Fig. 2, the crownbeams being repeatedly underpinned and supported by struts resting on intermediate crossbeams. These repeated underpinnings cause the major settlement, which with this method ranges generally from 20 to 80 cm., and sometimes much more, according to the size of the section and the nature of rock. Further settlement occurs by compression of the whole support construction as the load increases while the abutments are excavated.

When contemplating this process of loosening and development of load special consideration must be given to the time factor. To excavate the roof section

and the abutments stages *3* and *4*, of, say, an 8 m. length of a tunnel of 50 sq. m. section will ordinarily take one month. During this time the formation of natural arches inside the loosened rock is continuously disturbed by blasting vibration.

Even while the section is being lined the process of settling still continues, as the load has to be transferred gradually from the crownbeams, which are removed, to the masonry as it is carried towards the top. The lining of a section of the above-mentioned size takes a further 10 to 14 days. To concrete the same section today with modern methods would not even take the same number of hours.

It happens quite often that the roof has sunk too low for the prescribed thickness of lining to be inserted when arriving at the closure, and this part of the roof must consequently be laboriously re-excavated.

Not very long ago it was still customary to fill the overbreak with dry packing. This is a very bad system, particularly when used at the sides and in the lower part of the roof. The quality of execution under the unfavourable working conditions of poor lighting and hampered supervision is doubtful, and every dry packing, even a good one, is subject to a certain compression.

A further disadvantage of the Austrian method is that to a great extent the lagging has to be left in place, as its removal under load would cause a sudden loss of rock material and a dangerous increase of load. If the tunnel, when in service, happens not to be always below water level, the wood will decay after some time. The voids produced in this way give rise to serious disturbance of equilibrium and may finally cause the lining to deform by both decreasing the passive resistance and changing the load conditions.

To use rammed concrete instead of masonry in this case is not a good solution either, as frequent interruptions due to the removal of supporting elements are liable to leave many joints in the concrete, which are easily soiled. Moreover, it is very difficult to make such joints well radially, especially in the upper part of the arch. A vault made in rammed concrete will consequently always be far inferior to one concreted after modern principles.

To combine the Austrian method of excavation with modern uninterrupted concreting is not an easy task. In this case all the crownbeams must be removed before concreting is started and replaced by planks wedged against precast concrete blocks, the latter resting on the formwork.

There is no doubt that the heavy pressures that frequently developed when using this method were in the majority of cases more a consequence of the extremely unfavourable conditions of excavation with regard to loosening, as just described, than to the nature of the rock.

Steel arches. The method of support by steel arches, developed in the United States and increasingly favoured also in other countries, is incomparably better with regard to the reduction in the loosening of the rock. If this method of support is properly carried out, the arches are erected immediately behind the face. The ribs become stressed by wedging against the rock where it tends to move, and the outfall of rock between the arches is prevented by lagging. The loosening in this case proceeds in the following manner (see Fig. 3). After a round has been fired, the rock settles in the region of the advance t, being prevented from disintegrating by a double-arch effect, over the space of the advance as well as perpendicularly to the axis, and is supported additionally by crownbeams. The limits of this loosening will at first not go farther than $\frac{t}{2}$ as an unfavourable limit, provided the face is able to stand without bracing. A slight loosening may also take place inside the face and to a minor extent in the vertical portion of the side walls, although they may remain standing without support for a certain time. This will result in a deepening of the region of loosening, but in any case this will not go farther than t if steel arches are placed immediately and provided

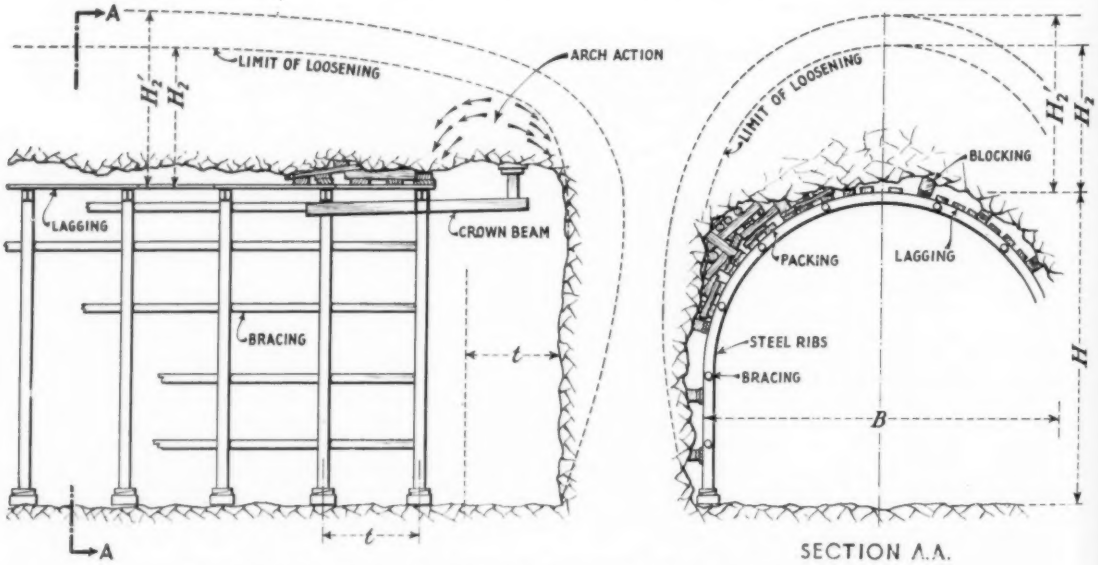

Fig. 3. Excavation with support by steel arches (full-face driving)

the walls do not need to be braced and the ratio $\frac{H}{B} \leq 1$. With heading and bench driving H is about halved and the loosening reduced accordingly. Further loosening will occur due to the inevitable voids between lagging and rock face. The magnitude of this loosening depends on the quality of packing and the material used. It can be minimised by using steel or concrete lagging with gravel blown behind.

Besides the yielding due to compression of wedges and packing, settlement of the groundplates which transfer the load from the arches to the ground must be taken into account. No further settling can be expected with full-face driving, but if, with heading and bench driving, the upper section is excavated first and the arches are placed on wallplates, some additional yield will occur at the points where the arches are supported on the bench and further movement will be caused by bending of the wallplates when underpinning them. The loosening conditions become somewhat worse when side galleries have to be driven first to lay the wallplates.

No further settling takes place during the lining of the tunnel as the arches are usually left in the concrete.

A short comparison between steel arching and the Austrian tunnelling method can be given as follows:

With steel arching the necessity of repeatedly unloading and resupporting the crownbeams, which causes the greatest settlement, does not arise. The resistance of steel to deformation is much better than that of wood, which starts to creep easily under load, particularly when soaked. There is an enormous difference in the time factor: to excavate and line a tunnel of the section cited above would take as many days with the steel-arch method as it takes weeks with the Austrian method. On the other hand, the fact that with steel arching the tunnel is not generally lined before the excavation is entirely finished will result in a minor increase of load. In exceptional cases this may necessitate the concreting beforehand of certain sections. Nevertheless this can be left out of consideration as it will not occur within the geological range of this article.

The problem of wood as lagging material exists of course also with the steel-arch method, though a smaller amount has to be left behind than with the Austrian method, as the beams are much less heavily loaded and a larger proportion of timber can consequently be retrieved. In any case the modern tendency is increasingly to reject wood as a lagging material in favour of steel or reinforced concrete. In the case of exceptionally large overbreak areas, backfilling with wood becomes dangerous, as the arch will deform, compressing the packing, and will probably fail. Concrete should be used in this case to fill the overbreak.

To give specific figures for the magnitude of load when the Austrian method is used we can refer to numerous results of tests carried out in many railroad tunnels in the eastern Alps. The loads were calculated from deformations of wooden support members of known strength.

For the geological range in question, according to these tests, the extent of the rock load H_1 on the roof ranges from $H_1 = 0{\cdot}5\ B$ to $1{\cdot}1\ (B + H)$.[3] This rather large range must of course be subdivided in order to design the temporary support structure (i.e., to determine the size and distance of timber sets or steel supports), but for the purpose of designing the lining a single average value of H_1 for each method of support will cover the whole range safely.

For the Austrian method an average $H_1 = B$ can be taken. For steel arching an average H_1 can range between $B/2$ and t. The first figure being recommended for unfavourable cases, i.e., heading and bench driving and poor packing and the second for full-face driving with careful blocking and lagging.

The influence of the height of the tunnel can be neglected for our purpose if, as is usually the case, the height does not exceed the width or does so only slightly.

Roofbolting. The third method to be taken into consideration is support by roofbolting[1] (see Fig. 4). As this system has already been described by the author in these pages* there is no need to go into details concerning this method, which enjoys rapidly increasing popularity. It need only be mentioned that it is possible to bolt nearly any kind of rock as long as a hole can be drilled into it and as long as the portion of roof exposed by blasting a round either stands up by itself for a couple of hours or can be supported temporarily from the drilling jumbo. Soft material which needs forepoling and perhaps even breasting is not suitable for bolting, the steel arch-method being more expedient for this case. It is, of course, very important to choose the right kind of bolt corresponding to the rock, using either slit-and-wedge or expansion-type bolts or concrete anchors.

By bolting the roof with the correct bolts in a proper manner the double natural-arch action, which arises automatically after blasting, is preserved. A bolted roof is consequently self-supporting. The loosening is considerably less even than with the steel-arch method, as the bolted regions are clamped against the sound rock by much greater forces than those obtainable by wedging, and practically no subsequent settling is possible.

Lagging is replaced by a net, assisted by channels connecting the bolts. Loose rock can be wedged against the channels or against some timber placed between them. There will consequently be some minor superficial loosening of rock between the bolting points, but this loosening is practically harmless as it is confined to small areas, and where it may seem expedient additional short bolts can easily be placed between the longer ones to impede local disintegration. With roofbolting, there is no question of any damage due to wooden lagging being left in the concrete. No other method is able to follow the irregularities of the excavation so closely. Packing is consequently either eliminated or reduced to some minor wedging.

If roofbolting is carried out correctly, i.e., simultaneously with drilling the next round, as close as possible to the face and using impact wrenches for tightening the nuts, the static equilibrium of the region will remain practically unchanged provided that no damage is done either to the rock or to the steel. Even this risk can be avoided by using concrete anchors, some types of which are so far developed that in certain cases they can be considered as a permanent support.[1]

Whereas the Austrian and the steel-arch method of excavation are unthinkable without a solid lining, concrete bolting can be a permanent support in itself

* "Bolted Support for Tunnels," WATER POWER, April 1954, p. 150, May 1954, p. 171

in suitable conditions, e.g., for a chemically intact coarsely jointed rock. If a net and a layer of gunite be added, the geological range of rock that can be supported permanently with concrete bolts is greatly widened. When bolting in shales or similar material this kind of support may become imperative in order to preserve the rock from deteriorating by seepage.

The extent of loosening with roofbolting is even smaller than with steel arching, amounting to about $H_1 = 0{\cdot}5\,t$ as an average, while the maximum possible loosening certainly cannot exceed the length of the bolts l.

At first the lining will receive no load at all, but later minor charges may arise, as the rock between the bolting points continues to loosen. Whether further additional load will develop or not depends on the kind of bolting. In the case of steel anchors the prestress in the rock will naturally only last until the bolts are corroded. Concreting in due time is imperative in this case. As the concreting can be regarded as being executed against a rock under prestress, once the bolts corrode the rock at the bolting points will transmit to the lining a force corresponding to that of the prestressing plus the weight of loosened rock. These forces lie approximately in the range of 5 to 8 tons per sq. m. But if the surrounding rock has been grouted soon after concreting, no slackening of prestressing will occur as the natural arch becomes transformed to an almost voidless permanent vault.

In case of concrete bolting no further forces will develop to place the lining under load, the load being limited to portions loosened between the anchors.

Effect of Grouting

When, decades ago, grouting was introduced in tunnelling maintenance to repair damaged linings the great advantages of this method became obvious, and today grouting is known to be of such fundamental importance in the lining of tunnels of all kinds that all modern specifications call for it.

The effect of grouting on stability is manifold: Voids between lining and rockface as well as joints, cracks and open spaces inside the surrounding rock are filled. This prevents loosening from progressing further and at the same time improves the modulus of elasticity of the material. The joints in the material which partakes in forming the natural arch are closed, so that the arch action is greatly improved and given a permanent character as a bearing member additional to the lining. Grouting places the lining under stress, which is highest at the time the grouting is carried out. In a soft rock the stress will diminish afterwards by slow plastic and elastic deformations of the rock material. With tunnels that are to receive inside pressures later, a certain prestressing effect is obtained.

Distribution and Properties of Load

The force exerted by loose rock upon the tunnel roof is $H_1\gamma$ tons per sq m. For γ, the value of solid rock should be taken. Depending on the method of driving, H_1 is constant over the whole excavated width for both the steel-arch and the roofbolting method, but for the Austrian method it is a maximum in the centre and decreases towards the sides, depending on the number of underpinnings and other features of the timbering and the rock.

The load H_1 stresses the permanent support structure, of which the concrete lining is only a part. The proportion in which temporary support members can be incorporated in the permanent structure depends on the method.

With the Austrian method the load of loose rock is transferred directly to the lining, which consequently becomes stressed as soon as the formwork is removed.

When supported by steel arches the whole load is carried by the ribs while the lining is concreted. The lining in this case can almost be considered as a mere means of preservation and strengthening. If wood is used as lagging and packing material the lining will become loaded after the packing has decayed. The

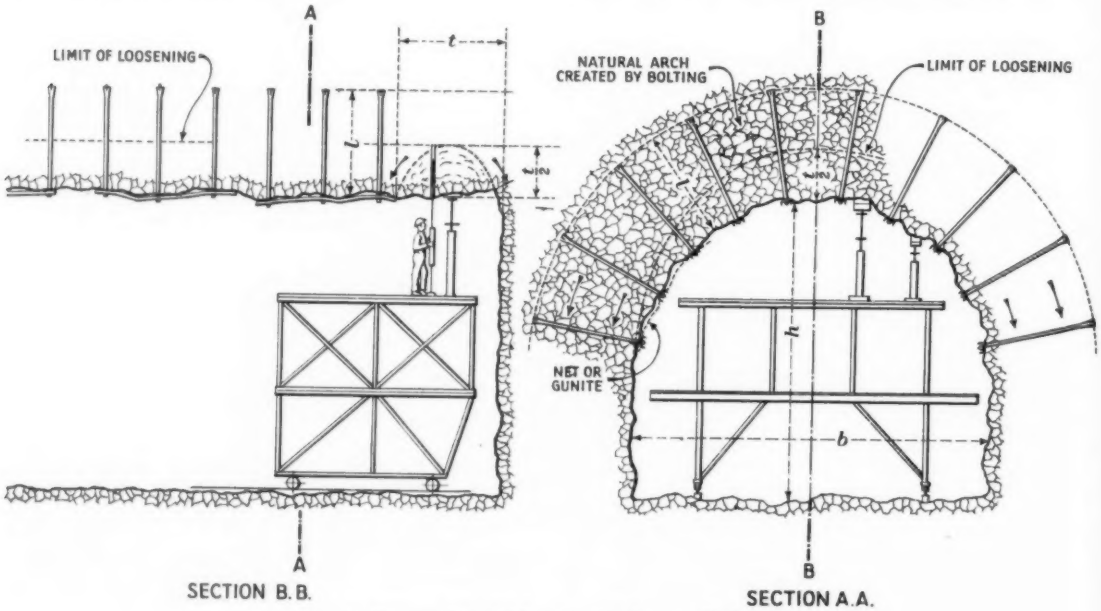

Fig. 4. Excavation with support by roofbolting

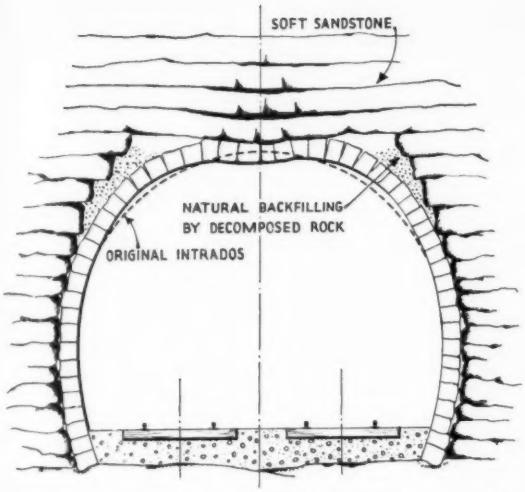

Fig. 5. Deformed double-track railroad tunnel in Czechoslovakia

steel arches will always act as valuable reinforcement even if they happen to be partially damaged in the course of time by rust, but this can be avoided by grouting the tunnel not too long after concreting. Forces caused by grouting will put the lining under stress.

With roofbolting the conditions are similar to those of steel arches. In this case the natural arch which is kept in place and strengthened by bolting preserves the excavation from caving in, and except for small superficial rock areas no load will impinge on the lining. In any case a lining is indispensable to allow the tunnel to be grouted and will probably receive its greatest load by grouting pressure. With roofbolting, permanent support consists of three essential parts of equal importance: the natural arch action, the concrete lining, and the grouting. The lining may be replaced by guniting in certain cases, which, together with concrete anchoring, is doubtless the most economic solution of a permanent lining.

Static Action of a Tunnel Arch

There is a fundamental difference between a free arch and that occurring in a tunnel. A free arch will deform under load, but in a tunnel made after modern principles the arch cannot yield. When designing an outside arch to be made of concrete or masonry, care should be taken to keep the thrust line well inside the middle third of the section in order to avoid tension cracks. On the contrary, with a tunnel arch which cannot give way, the thrust line will almost coincide with the axis of the section, independently of the form of the arch, minor deflections due to small elastic deformations by temperature, yield and load being insignificant.[5]

A lining in rock within the geological range we are considering can only deform perceptibly due to the presence of voids immediately surrounding. Such cases of deformation were unusual even in earlier times, as the thickness of linings was generally overdone, and dry packing and loosened rock became compacted to a great extent by the weight of superposed rock, which resulted in an increase in the passive resistance of the material.

The author happened by chance to see a typical example of this kind of deformation in a two-track railway tunnel on a main line in Czechoslovakia. The tunnel is rather shallow and runs through soft sandstone bedded horizontally, with thin intersections of clay and slightly waterbearing.[4] Obviously the tunnel did not need much temporary support when it was excavated. The overhanging strata on both sides probably broke down, leaving a more or less rectangular section with a horizontal roof. When the tunnel was lined immediately afterwards with a masonry arch 35 cm. thick nicely made of properly cut stones, no care was taken of the backfill, and actually nothing was done at all to fill the large voids between the arch and the rock. When the author saw the tunnel 50 years after construction it was in a rather dangerous state, as the intrados were deformed to a slightly convex line at the top, and the lower parts of the arch consequently yielded to the sides. A gallery driven above the top of the arch to inspect it, clearly disclosed the reason for the serious deformation. In the course of time the upper layers of the sandstone had sunk down until they reached the top of the extrados, pressing the arch down and causing it to deform (see Fig. 5). It was interesting that the tunnel had not failed long before, but served half a century without repair and still was in full use at the time of inspection. This was obviously due to some automatic natural backfill from the side walls, out of which some weathered rock dropped in the course of time, preventing the tunnel from deforming to the point of failure. This would, of course, never happen with a tunnel built after modern principles. The example is nevertheless a typical illustration of the rule that a tunnel never deforms by loosening pressure unless a grave blunder has been committed during the construction.

(To be continued)

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Savigliano. La Società Nazionale delle Officine di Savigliano, of Turin, is one of the oldest industrial concerns in Piemonte having been established as a joint stock company since 1880. Two admirably produced catalogues received from them give an illustrated review of their products, a predominant proportion of which consists of plant used in hydroelectric schemes, viz., sluice gates, valves, high-pressure pipelines and penstocks, electric generators and transformers, transmission lines, steel bridges, cranes, tractors and road-making equipment. Both publications consist largely of photographic views which are captioned in both Italian and English, but there are short introductions in English in each case. Extensive lists of power stations equipped with Savigliano sluice gates, high-pressure pipelines, generators, transformers, etc., are given in the form of appendices.

An extension of the theory of
Water Hammer

The substance of a paper read by R. Skalak, Assistant Professor of Civil Engineering, Columbia University, before the Diamond Jubilee Spring Meeting of the American Society of Mechanical Engineers

PART ONE

THE usual theory of water hammer which is used to compute the magnitude and velocity of pressure waves in a pipe filled with an elastic fluid was given by Joukowsky (Ref. 8) in 1898. This theory predicts that pressure waves travel without change of shape at a velocity c_e:

$$c_e = \frac{c}{\sqrt{1+\frac{2Ka}{Eh}}} \qquad \ldots(1)$$

In the derivation of this formula it is assumed that the pressure is uniform across any section of the pipe and that the deflection of the pipe is equal to the static deflection due to the instantaneous pressure in the fluid. The first assumption neglects, in effect, the inertial forces associated with radial motion of the fluid. The second neglects the mass of the pipe wall and the longitudinal and bending stresses in the pipe wall.

For the case of an infinite train of sinusoidal pressure waves in a tube filled with fluid, much work has been done without resorting to the above assumptions. In 1878 Korteweg (Ref. 9) showed that the phase velocity of sinusoidal waves varies with the wave-length when the radial inertia of the fluid and the mass of the pipe wall are taken into account. Korteweg also gave the expression for c_e, eq. (1) as an approximation of the phase velocity for long wave-lengths. The work of Korteweg was extended in 1898 by Lamb (Ref. 10) who considered the pipe as an elastic membrane thus including the effect of longitudinal stresses in the pipe wall. Lamb showed there are two finite phase velocities as the wave length approaches infinity. One of these is very close to Korteweg's result c_e and the other is very nearly the velocity of sound in the material of the tube wall. Recent papers by Jacobi (Ref. 6) and Thomson (Ref. 14) include the flexural stiffness of the pipe wall. The inclusion of the bending stresses affects the phase velocities appreciably only for very short wave-lengths.

In the present paper water-hammer waves are considered without some of the simplifications made in the Joukowsky theory. The equations of motion for a thin cylindrical tube are used, *cf.* Flügge (Ref. 11), with one additional term for the effect of rotatory inertia. The fluid is assumed to be elastic and inviscid but no assumptions are made regarding its motion except that it is small. Only the case where the velocity of sound in the material of the pipe wall is

NOMENCLATURE

The following nomenclature is used in the paper:

c	Velocity of sound in fluid
c_e	Joukowsky water hammer velocity (See eq. 1)
K	Bulk modulus of fluid
a	Radius of tube
h	Thickness of tube wall
E	Young's modulus of tube wall material
p	Pressure in fluid
v	Axial velocity of fluid
z	Distance axially along the tube
t	Time
u	Axial displacement of tube
w	Radial deflection of tube wall (+outward)
ρ_0	Initial density of fluid
r	Radial distance
ϕ	Velocity potential
m	Mass of tube per unit surface area
ν	Poisson's ratio for tube wall
c_0	$\sqrt{\frac{Eh}{m(1-\nu^2)}}$ Velocity of sound in tube wall
q	Force per unit area on tube wall
ξ, Ω	Transform variables
n, s	Integers
N	See eq. (23)
Δ	See eq. (24)
J_0, J_1	Bessel functions
i	$= \sqrt{-1}$
D	See eq. (30)
F	See eq. (36)
G	See eq. (37)
B	See eq. (38)
$c', \bar{c}$	Arbitrary constant velocities
c_1, c_2	Phase velocities
z'	See eq. (48)
η	Variable of integration
β	See eq. (50)
I, H	Integrals. See eq. (53)
L	Length of wave front
R	$= \frac{c_0^2}{c^2}$
A	$= \frac{\rho_0 a}{m}$

greater than in the fluid will be considered. Much of the analysis would be valid in the opposite case, but the relative speeds of the various waves would differ.

A Formal Solution by Integral Transforms

In this section a formal solution of a typical water-hammer problem is obtained in the form of a double integral. The solution is reduced to terms of single real integrals whose evaluation is discussed in the next section. The problem considered is to find the motion of a thin cylindrical tube filled with fluid starting from given initial conditions. The conditions, shown in Fig. 1, are that for $t = 0$:

$$\begin{aligned} &p = 0,\ v = 0 \text{ for } z>0 \\ &p = p_0,\ v = v_0 \text{ for } z<0 \\ &u = 0,\ w = 0, \text{ for all } z \end{aligned} \qquad \ldots(2)$$

The velocity v_0 is assumed to be related to the pressure p_0 by:

$$v_0 = \frac{\rho_0}{\rho_0 c} \qquad \ldots(3)$$

This is the velocity that would exist behind a plane shock wave of pressure p_0 in an infinite body of fluid. Thus the initial conditions for the fluid correspond to a step shock wave moving in the positive direction. An external pressure equal to p_0 is assumed acting on the outside of the tube for $z<0$ to prevent any motion of the tube at $z = -\infty$. This external pressure is assumed to remain on the tube for $z<0$ for all t.

These initial conditions are intended to simulate the situation arising when a pipe leading from a reservoir is filled with fluid at rest below reservoir pressure and a valve at the reservoir end is suddenly opened. The part of the tube left of the origin in Fig. 1 is a convenient device for supplying fluid and energy in place of a reservoir. It is expected that the particular means of generating a water-hammer wave will not essentially affect its characteristics when the wave has progressed far enough down the pipe.

The motion of the fluid will be described by a velocity potential which must satisfy the equation :

$$\nabla^2\phi = \phi_{rr} + \frac{1}{r}\phi_r + \phi_{zz} = \frac{1}{c^2}\ddot{\phi} \qquad \ldots(4)$$

where the subscripts r and z denote derivatives.

The motion of the tube due to a radial force per unit area, $q\ (z,\ t)$, is governed by two equations :

$$\frac{\ddot{w}}{c_0^2} - \frac{h^2}{12c_0^2}\ddot{w}_{zz} + \left(1 + \frac{h^2}{12a^2}\right)\frac{w}{a^2} + \frac{\nu}{a}u_z + \frac{h^2}{12}w_{zzzz} - \frac{h^2}{12a}u_{zzz} = \frac{q}{mc_0^2} \qquad \ldots(5)$$

$$\frac{\ddot{u}}{c_0^2} - u_{zz} - \frac{\nu}{a}w_z + \frac{h^2}{12a}w_{zzz} = 0 \qquad \ldots(6)$$

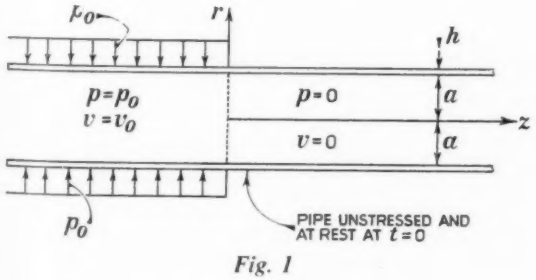


Fig. 1

A detailed derivation of these equations is given in (Ref. 12). They agree with the equations given by Flügge (Ref. 11) except that the rotatory inertia term $\ddot{w}_{zz}$ has been added above.

The fact that the fluid and tube are expected to remain in contact is expressed by :

$$\dot{w} = -\left[\frac{\partial\phi}{\partial r}\right]_{r=a} \qquad \ldots(7)$$

The pressure exerted on the tube wall by the fluid is :

$$\left[p\right]_{r=a} = \rho_0\left[\frac{\partial\phi}{\partial t}\right]_{r=a} \qquad \ldots(8)$$

This is the value of q to be used for $z > 0$ in eq. (5). For $z < 0$, the external pressure $-p_0$ must be added.

It is convenient to consider the solution of the problem as the sum of two parts :

The first part is the shock wave that would be produced by the initial conditions if the tube were rigid. The tube displacements w_1, u_1, for this part are zero. The velocity and pressure v_1, p_1, for the fluid are :

$$\left.\begin{aligned} v_1 &= v_0 \\ p_1 &= \rho_0 \end{aligned}\right\} \text{ for } z < ct \qquad \left.\begin{aligned} v_1 &= 0 \\ p_1 &= 0 \end{aligned}\right\} \text{ for } z > ct \qquad \ldots(9)$$

The second part is the response of the tube and fluid to a progressive load pressure p_s equal to the pressure produced by the shock wave of the first part. This load pressure acts on the tube wall radially outward. Its magnitude is p_0 for $0 < z < ct$ and zero elsewhere.

Inasmuch as w_1, u_1, v_1, etc. are known, the problem reduces to finding the response w_2, u_2, v_2, etc. of the tube and fluid to the load pressure p_s. For this second part of the solution the system is initially at rest and the governing equations are (4), (5), (6), (7) with:

$$q_2 = \rho_0\left[\frac{\partial\phi}{\partial t}\right]_{r=a} + p_s \qquad \ldots(10)$$

The solution is begun by transforming the governing equations and eliminating the velocity potential ϕ_2. The remaining transformed equations are solved simultaneously and the inversions of the transformations then yield the solution.

With respect to z a Fourier transform (*cf.* Ref. 15, p. 42) is used. Defining the transform of a function $g(z,t)$ with respect to z by :

$$\bar{g}(\xi,t) = \frac{1}{2\pi}\int_{-\infty}^{\infty} g(z,t)e^{-i\xi z}dz \qquad \ldots(11)$$

then the inversion is :

$$g(z,t) = \int_{-\infty}^{\infty} \bar{g}(\xi,t)e^{i\xi z}d\xi \qquad \ldots(12)$$

With respect to time a Laplace transform is used, but the transform variable adopted is $-i = -\sqrt{-1}$ times the usual transform variable. This results in a certain symmetry with the first transform and convenience in later analysis. Defining the transform of a function $\bar{g}(\xi,t)$ with respect to t by :

$$g(\xi,\Omega) = \frac{1}{2\pi}\int_{0}^{\infty} \bar{g}(\xi,t)e^{-i\Omega t}dt \qquad \ldots(13)$$

then the inversion is :

$$\bar{g}(\xi,t)=\int_{-\infty-i\alpha}^{\infty-i\alpha}\bar{g}(\xi,\Omega)e^{i\Omega t}d\Omega \qquad ...(14)$$

The double transform of derivatives of $g(z,t)$ may be expressed in terms of the double transform of $g(z,t)$ by the usual partial integration procedure (Ref. 13, p. 27). Using the fact that initial velocities and displacements are zero, the transforms of the derivatives involved are :

$$\overline{\frac{\partial^n}{\partial z^n}\frac{\partial^s}{\partial t^s}\left[g(z,t)\right]}=(i\xi)^n(i\Omega)^s\bar{g}(\xi,\Omega) \qquad ...(15)$$

Using q_2 as given by eq. (10), the double transformation of eqs. (4), (5), (6) and (7) yields :

$$\bar{\phi}_{2rr}+\frac{1}{r}\bar{\phi}_{2r}-\xi^2\bar{\phi}_2=-\frac{\Omega^2}{c^2}\bar{\phi}_2 \qquad ...(16)$$

$$-\frac{\Omega^2}{c_0^2}\bar{w}_2-\xi^2\Omega^2\frac{h^2}{12c_0^2}\bar{w}_2+\left(1+\frac{h^2}{12a^2}\right)\frac{\bar{w}_2}{a^2}+i\xi\frac{\nu}{a}\bar{u}_2+$$

$$+\frac{\xi^2h^4}{12}\bar{w}_2+i\xi^3\frac{h^2}{12a}u_2=i\Omega\frac{\rho_0}{mc_0^2}\left[\bar{\phi}_2\right]_{r=a}+\frac{\bar{p}_s}{mc_0^2} \qquad ...(17)$$

$$-\frac{\Omega^2}{c_0^2}u_2+\xi^2u_2-i\xi\frac{\nu}{a}\bar{w}_2-i\xi^3\frac{h^2}{12a}\bar{w}_2=0 \qquad ...(18)$$

$$i\Omega\bar{w}_2=-\left[\bar{\phi}_{2r}\right]_{r=a} \qquad ...(19)$$

The double transforms are functions of ξ and Ω only except for ϕ_2 which is $\phi_2(\xi,\Omega,r)$. From eq. (16) it is found that the r dependence of $\bar{\phi}_2$ involves a Bessel function of the first kind. The function p_s is known. Solving eqs. (16), (17), (18), (19) for $\bar{w}_2$, $\bar{u}_2$, $\bar{\phi}_2$ and inverting yields :

$$w_2=-\frac{p_0}{4\pi^2m}\int_{-\infty}^{\infty}e^{i\xi z}d\xi\int_{-\infty-i\alpha}^{\infty-i\alpha}\frac{e^{i\Omega t}}{\Omega^3\left(\frac{\Omega}{c}+\xi\right)\left[\frac{\rho_0a}{2m}\Delta+\frac{1}{N}\right]}d\Omega \qquad ...(20)$$

$$u_2=\frac{p_0}{4\pi^2m}\int_{-\infty}^{\infty}e^{i\xi z}\times$$

$$\times d\xi\int_{-\infty-i\alpha}^{\infty-i\alpha}\frac{vi\xi\left(1+\frac{h^2}{12\nu}\xi^2\right)e^{i\Omega t}}{a\left(\frac{\Omega^2}{c_0^2}-\xi^2\right)\Omega^3\left(\frac{\Omega}{c}+\xi\right)\left[\frac{\rho_0a}{2m}\Delta+\frac{1}{N}\right]}d\Omega \qquad ...(21)$$

$$\phi_2=-\frac{p_0}{4\pi^2m}\int_{-\infty}^{\infty}e^{i\xi z}\times$$

$$\times d\xi\int_{-\infty-i\alpha}^{\infty-i\alpha}\frac{iJ_0\left(r\sqrt{\frac{\Omega^2}{c_0^2}-\xi^2}\right)e^{i\Omega t}\,d\Omega}{\Omega^2\sqrt{\frac{\Omega^2}{c^2}-\xi^2}\,J_1\left(a\sqrt{\frac{\Omega^2}{c^2}-\xi^2}\right)\left(\frac{\Omega}{c}+\xi\right)\left[\frac{\rho_0a}{2m}\Delta+\frac{1}{N}\right]} \qquad ...(22)$$

where :

$$\frac{1}{N}=-1+\frac{\Omega^2-c_0^2(1-\nu^2)\xi^2}{\Omega^2a^2\left(\frac{\Omega^2}{c_0^2}-\xi^2\right)}+\frac{h^2}{12}\left[-\xi^2+\frac{c_0^2}{a^4\Omega^2}+\right.$$

$$\left.+\frac{2c_0^2\nu\xi^4}{\Omega^2a^2\left(\frac{\Omega^2}{c_0^2}-\xi^2\right)}+\frac{c_0^2\xi^4}{\Omega^2}+\frac{c_0^2h^2\xi^6}{12\Omega^2a^2\left(\frac{\Omega^2}{c_0^2}-\xi^2\right)}\right] \qquad ...(23)$$

$$\Delta=\frac{2J_0\left(a\sqrt{\frac{\Omega^2}{c^2}-\xi^2}\right)}{a\sqrt{\frac{\Omega^2}{c^2}-\xi^2}\,J_1\left(a\sqrt{\frac{\Omega^2}{c^2}-\xi^2}\right)} \qquad ...(24)$$

The evaluation of the radial deflection w_2 will be considered first. It is typical of the quantities of interest. The inner or Ω integral of eq. (20) may be evaluated by Cauchy's residue theorem applied in the complex Ω plane. During this process ξ is regarded as a constant real parameter since the outer integral involves only real values of ξ.

The factor Ω^3 in the denominator of the integrand in eq. (20) produces a simple pole at $\Omega=0$ because $\Omega^2(1/N)$ is finite at $\Omega=0$. There is no pole at $(\Omega/c+\xi)=0$ because $(\Omega/c+\xi)\Delta$ is finite at this point. There are poles at the points for which the remaining bracket is zero, i.e., where :

$$\left[\frac{\rho_0a}{2m}\Delta+\frac{1}{N}\right]=0 \qquad ...(25)$$

For any real ξ this equation has an infinite number of roots Ω_n. These roots have an interesting physical significance. They are the circular frequencies of the modes of free vibration of the tube and fluid system having a wave-length $\lambda=2\pi/\xi$. Alternatively the ratios Ω_n/ξ are the phase velocities c_n of progressive sinusoidal waves of wave-length $2\pi/\xi$. These facts may be developed by considering the governing equations (4), (5), (6) and (7) with q in eq. (5) equal to the fluid pressure only. A solution is assumed of the form :

$$\left.\begin{aligned}w&=w'e^{i(\xi z+\Omega t)}\\u&=u'e^{i(\xi z+\Omega t)}\\\phi&=\phi'f(r)e^{i(\xi z+\Omega t)}\end{aligned}\right\} \qquad ...(26)$$

where the prime quantities are constants and $f(r)$ is an unknown function of r. Substituting eq. (26) in eqs. (4), (5), (6) and (7) the constants and $f(r)$ may be eliminated, leading to exactly the transcendental eq. (25) relating ξ and Ω.

For real values of ξ all the roots Ω_n of eq. (25) are also real. If any of the Ω_n were complex or imaginary the corresponding free vibration would grow or decay

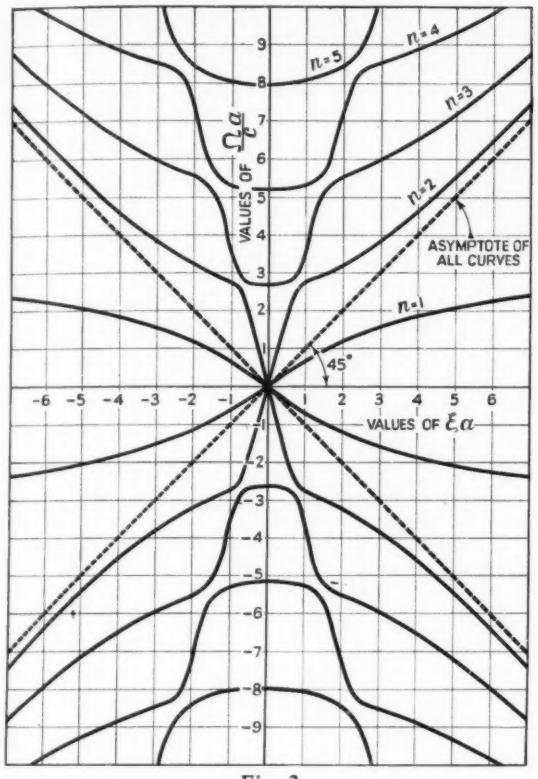


Fig. 2

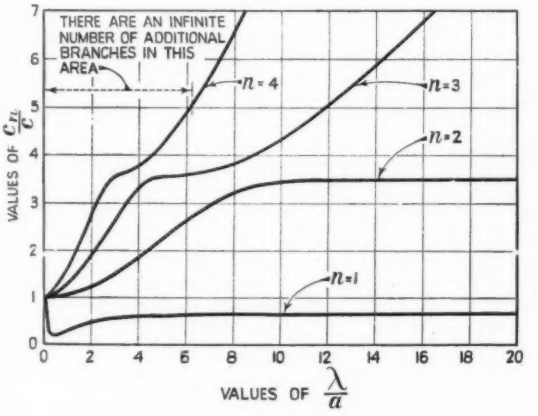


Fig. 3

indefinitely in time. Since the system has no means of dissipating or radiating energy, this is not possible physically.

Fig. 2 shows the Ω_n *vs* ξ values for a typical case in which the velocity of sound in the tube wall is greater than in the fluid, i.e., $c_0 > c$. It is found that for any value of ξ there is one root Ω_1, for which $(\Omega^2/c^2-\xi^2)$ is negative. This root corresponds to a phase velocity less than c, the velocity of sound in the fluid. There are also an infinite number of roots for which $(\Omega^2/c^2-\xi^2)$ is positive. These correspond to phase velocities greater than c. Fig. 3 shows the same data as in Fig. 2 plotted as phase velocity *vs* wave-length.

The fact that two sets of Ω_n curves pass through the origin in Fig. 2 shows that there are two modes which have finite phase velocities as the wave length approaches infinity and the frequency approaches zero. This was also shown by Lamb (Ref. 10, p. 7). It is mentioned here because in a recent paper (Ref. 14, p. 933), it is erroneously stated that there is a cutoff frequency below which only one mode may be propagated.

The above discussion shows that the poles of the integrand of eq. (20) are at $\Omega = 0$ and $\Omega = \Omega_n$, all on the real Ω axis. The path of integration is below this axis and it will be closed in the usual manner by adding a semicircle of infinite radius in the upper half of the Ω plane. The integral over the semicircle approaches zero as the radius approaches infinity. The contribution of the pole at $\Omega =$ to w_2 is :

$$w_{2a} = \frac{p_0}{2\pi i m} \times$$

$$\int_{-\infty}^{\infty} \frac{e^{i\xi z}}{\xi\left[\frac{c_0^2}{a^2}\left(1-\nu^2+\frac{h^2}{12a^2}\right)-\frac{c_0^2\nu h^2}{6a^2}\xi^2+\frac{h^2 c_0^2}{12}\left(1-\frac{h^2}{12a^2}\right)\xi^4\right]} d\xi \qquad \ldots(27)$$

The contributions of the Ω_n poles to w_2 may be expressed as :

$$w_{2b} = -\frac{p_0}{4\pi^2 m}\int_{-\infty}^{\infty} e^{i\xi z} \times$$

$$\times 2\pi i \sum_{\Omega_n}\left[\frac{e^{i\Omega t}}{\Omega^3\left(\frac{\Omega}{c}+\xi\right)\frac{d}{d\Omega}\left[\frac{\rho_0 a}{2m}\Delta+\frac{1}{N}\right]}\right]_{\Omega=\Omega_n} d\xi \qquad \ldots(28)$$

The sum indicated is over the Ω_n roots for each value of ξ. The values of the Ω_n as functions of ξ are given by Fig. 2. Eq. (28) may be written in the form

$$w_{2b} = \frac{-p_0}{2\pi i m}\int_{-\infty}^{\infty}\sum_{\Omega_n}\frac{e^{i(\xi z+\Omega t)}}{\left(\frac{\Omega}{c}+\xi\right)D} d\xi \qquad \ldots(29)$$

where:

$$D = -\Omega^3\frac{d}{d\Omega}\left[\frac{\rho_0 a}{2m}\Delta+\frac{1}{N}\right] = \Omega^3\left\{\frac{\rho_0\Omega a}{mc^2\left(\frac{\Omega^2}{c^2}-\xi^2\right)}\times\right.$$

$$\times\left[1+\frac{J_0^2}{J_1^2}\right]+\frac{2c_0^2}{\Omega^3 a^2}\left[\frac{\nu^2}{\left(1-c_0^2\frac{\xi^2}{\Omega^2}\right)^2}+1-\nu^2+\right.$$

$$\left.\left.+\frac{h^2(1+a^4\xi^4)}{12a^2}+\frac{h^2\xi^4\left(2\frac{\Omega^2}{c_0^2}-\xi^2\right)}{12\left(\frac{\Omega^2}{c_0^2}-\xi^2\right)^2}\left(2\nu+\frac{h^2\xi^2}{12}\right)\right]\right\} \qquad \ldots(30)$$

The integral in eq. (29) covers all four quadrants of

the Ω_n curves in Fig. 2, but it may be expressed as an integral over the first quadrant values only because D is even in Ω and even in ξ and the Ω_n curves are symmetric about both axes.
Thus :

$$w_{2b} = \frac{p_0}{\pi m}\int_0^\infty \sum_{\Omega_n}\left[\frac{\sin(\xi z-\Omega t)}{\left(\frac{\Omega}{c}-\xi\right)D} - \frac{\sin(\xi z+\Omega t)}{\left(\frac{\Omega}{c}+\xi\right)D}\right]d\xi \qquad \ldots(31)$$

where the summation is over the Ω_n curves in only the first quadrant of Fig. 2. This concludes the formal solution for w. The evaluation of the integrals remaining in eqs. (27) and (31) will be considered in the next section.

Instead of evaluating u_2 and ϕ_2, certain quantities of interest which are derivatives of u_2 and ϕ_2 will be computed directly. These are :

$$\left.\begin{aligned}&\text{Pressure in the fluid :} && p_2 = \rho_0\frac{\partial\phi_2}{\partial t}\\ &\text{Axial velocity of the fluid :} && v_2 = -\frac{\partial\phi_2}{\partial t}\\ &\text{Longitudinal strain in the tube :} && u_{z2} = \frac{\partial u_2}{\partial z}\end{aligned}\right\} \ldots(32)$$

Expressions for p_2, v_2 and u_{z2} are derived from the double integrals for u_2 and ϕ_2, eqs. (21) and (22), through application of eq. (15). The inner integral in Ω may be evaluated in each case by using Cauchy's residue theorem as in the case of w_2. The results are :

$$p_2 = \frac{p_0}{2\pi i}\int_{-\infty}^{\infty}\frac{e^{i\xi(z-ct)}}{\xi}\,d\xi + \frac{p_0\rho_0}{\pi m}\int_0^\infty \sum_{\Omega_n}\left[\frac{\sin(\xi z-\Omega t)}{\left(\frac{\Omega}{c}-\xi\right)F} - \frac{\sin(\xi z+\Omega t)}{\left(\frac{\Omega}{c}+\xi\right)F}\right]d\xi \qquad \ldots(33)$$

$$v_2 = \frac{p_0}{2\pi i\rho_0 c}\int_{-\infty}^{\infty}\frac{e^{i\xi(z-ct)}}{\xi}\,d\xi + \frac{p_0}{\pi m}\int_0^\infty \sum_{\Omega_n}\left[\frac{\sin(\xi z-\Omega t)}{\left(\frac{\Omega}{c}-\xi\right)G} + \frac{\sin(\xi z+\Omega t)}{\left(\frac{\Omega}{c}+\xi\right)G}\right]d\xi \qquad \ldots(34)$$

$$u_{z2} = \frac{ip_0\nu}{2\pi ma}\int_{-\infty}^{\infty}\frac{\left(1+\frac{h^2\xi^2}{12\nu}\right)e^{i\xi z}}{f(\xi)}\,d\xi + \frac{p_0\nu}{\pi ma}\int_0^\infty \sum_{\Omega_n}\left[\frac{\sin(\xi z-\Omega t)}{\left(\frac{\Omega}{c}-\xi\right)B} - \frac{\sin(\xi z+\Omega t)}{\left(\frac{\Omega}{c}+\xi\right)B}\right]d\xi \qquad \ldots(35)$$

where :

$$F = -\frac{D}{\Omega^2}\sqrt{\frac{\Omega^2}{c^2}-\xi^2}\,\frac{J_1\left(a\sqrt{\frac{\Omega^2}{c^2}-\xi^2}\right)}{J_0\left(r\sqrt{\frac{\Omega^2}{c^2}-\xi^2}\right)} \qquad \ldots(36)$$

$$G = F\frac{\Omega}{\xi} \qquad \ldots(37)$$

$$B = +\frac{D\left(\frac{\Omega^2}{c_0^2}-\xi^2\right)}{\xi^2\left(1+\frac{h^2}{12\nu}\xi^2\right)} \qquad \ldots(38)$$

and $f(\xi)$ stands for the denominator of the integrand in eq. (27).

The solution of the problem originally posed at the beginning of this section is given formally by :

$$\left.\begin{aligned} w &= w_{2a}+w_{2b} && \text{Eqs. (27) and (31)}\\ p &= p_1+p_2 && \text{Eqs. (9) and (33)}\\ v &= v_1+v_2 && \text{Eqs. (9) and (34)}\\ u_z &= u_{z2} && \text{Eq. (35)}\end{aligned}\right\} \ldots(39)$$

(To be continued)

The BEAMA Catalogue

There could be no better single source of information about the vast British electrical engineering industry than the new third edition of the Catalogue of the British Electrical and Allied Manufacturers' Association, introduced at a recent Press Conference by the Rt. Hon. The Viscount Chandos, D.S.O., M.C., the president of BEAMA. Weighing over 10 lb., and having more than 1,000 pages, the Catalogue is divided into seven sections. The first three of these form respectively comprehensive lists of firms engaged in the manufacture of power plant; equipment for industry, transport and communication; and commercial and domestic equipment. Each firm's entry gives a wealth of technical detail about its products. A buyers' guide and a trade directory are among the other useful features, and an especially valuable aspect is the provision of a glossary in English, French, German, Portuguese and Spanish. Some 15,000 copies are being sent out to reach suitable recipients in every country in the world.

The electrical industry is now Britain's second largest exporting industry, the value of its goods sent overseas in 1954 exceeding £220 million. The new BEAMA Catalogue, besides assisting in the British electrical export activities, is a volume of immense value as a comprehensive reference book.

Industrial Instrumentation. A brochure just received from the Hagan Corporation, Pittsburgh 30, Pa., gives some indication of the wide experience of this well-known firm in the field of automatic control and industrial instrumentation.

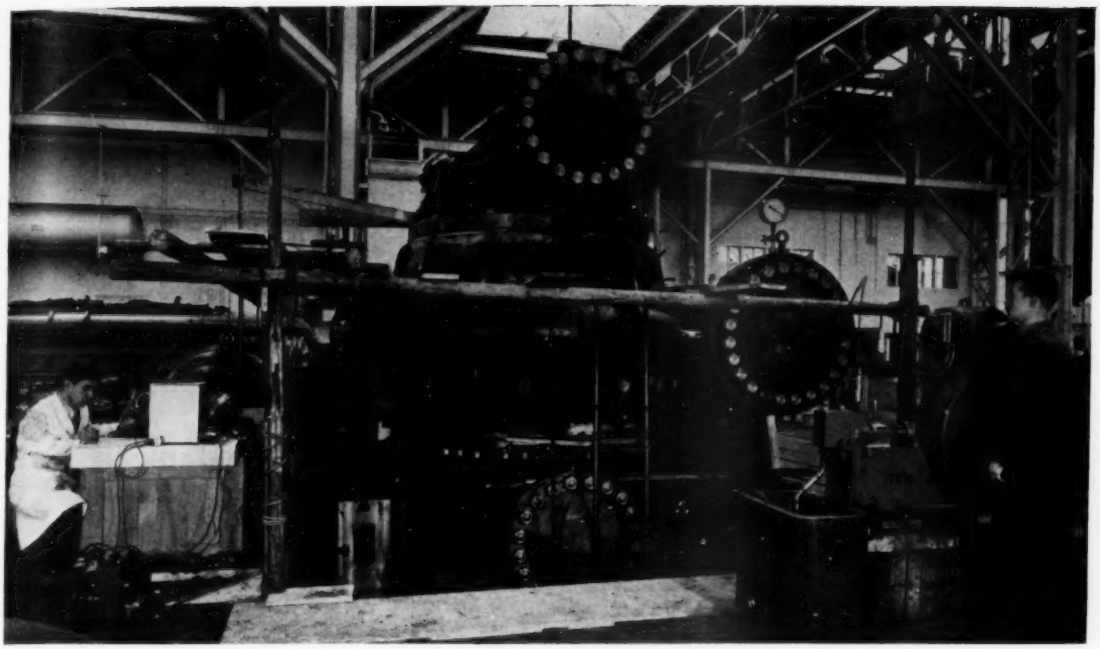


Fig. 1. Casing of the Grimsel-Oberaar high-lift storage pump on the test-bed. On the left is the strain-measuring equipment, and on the right the high-lift test pump with calibrated pressure gauge

Strain-Gauge Measurements on High-Lift Storage Pumps

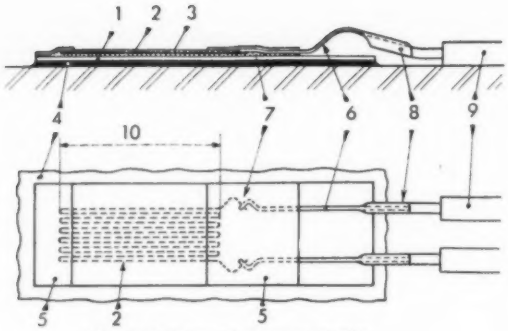
Since the introduction of the wire-resistance strain gauge for measuring stresses in engineering components, Sulzer Brothers have carried out numerous tests on pump casings and covers with this device and have thereby acquired information of great value in the calculation and designing of components for high-lift pumps. After a short description of the method, this articles gives an account of the measurements made on the storage pump for the Grimsel-Oberaar power station, Switzerland

By L. MARTINAGLIA

THE complicated shape of many machine components makes it extremely difficult or even impossible to calculate their strength with any accuracy, and the mathematical handling of such problems is in itself often most troublesome and time-consuming. As a consequence the stresses in such components have frequently to be determined by practical measurements, and successful development work in modern engineering would in fact be unthinkable without the aid of investigations of this kind. They provide the designer with the information he needs for perfecting his components, or they confirm the accuracy of his initial calculations—in short, they point the way to reliable and economical designs.

In recent years practical measuring methods—often themselves complicated enough—have been extended by the introduction of a new, simple and universal device: the wire-resistance strain gauge, commonly known simply as the "strain gauge."

The strain gauge turns to account the change in the electric resistance of thin wires when elastically deformed. If a metal wire is put under tensile load, it



1 Carrier foil (special paper or plastic).
2 Resistance wires.
3 Layer of adhesive.
4 Glue between gauge and metal.
5 Paper protecting ends of wire.
6 Lead-in wires.
7 Welded junction.
8 Soldered junction
9 Leads to Wheatstone bridge.
10 Gauge length.

Fig. 2. Side elevation and plan of a strain gauge of conventional design, about actual size (thickness in side elevation greatly exaggerated)

stretches, and at the same time its cross-section is reduced. These geometrical changes in themselves affect the electric resistance. In addition, measurements have shown that the specific resistance of the metal is also influenced by the stress conditions set up in it. The specific resistance of some alloys changes considerably when they are lengthened or shortened, the change being proportional to the change in length. Wires of these materials are therefore highly suitable for use in strain gauges.

The strain gauges commercially available today are mass produced. They consist of a very thin resistance wire which is bent in the form of a closely spaced grid and attached to a carrier usually consisting of thin rice paper or occasionally of a plastic material (Fig. 2). Two thicker wires welded to the ends of the resistance wire are used for supplying the electric current. The small, self-contained measuring element thus formed is stuck on the component with a special adhesive at the place where it is desired to measure the local strain, after the fashion of a postage stamp. Fig. 3 shows a number of types of strain gauges together

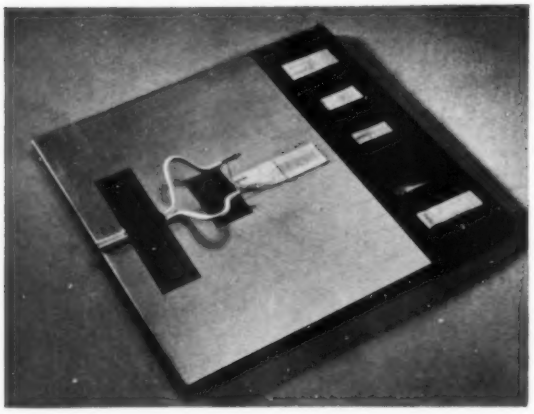
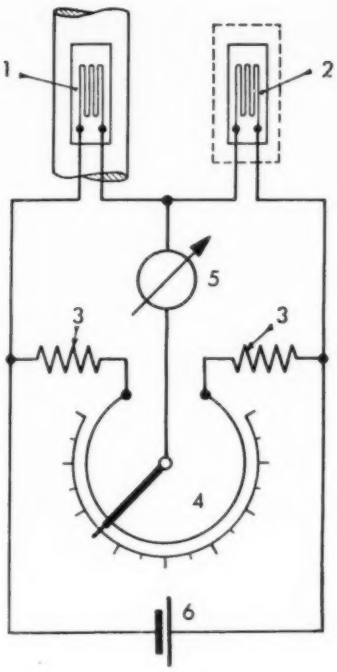


Fig. 3. A selection of commercial strain gauges, with one complete measuring rig



1 Working gauge (on component).
2 Dummy gauge (on unstressed piece of same material).
3 Compensating resistances of bridge.
4 Slide-wire resistance with calibrated scale.
5 Galvanometer.
6 Source of electromotive force.

Fig. 4. Diagram of a Wheatstone bridge for measuring strain under static stress

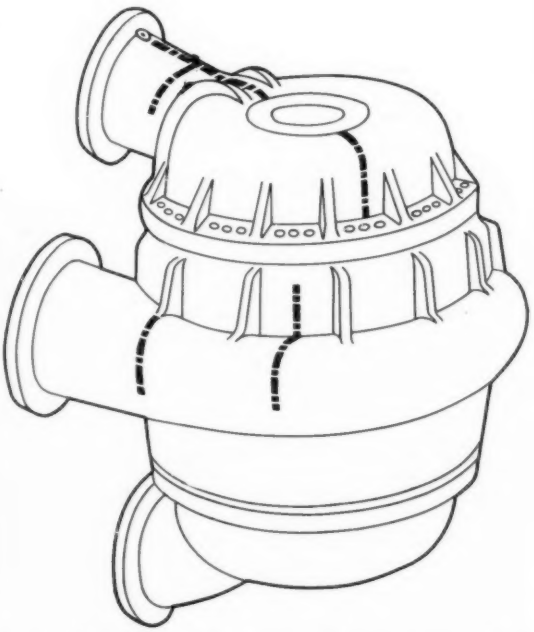


Fig. 5. Location of the sections of measurement on the casing of the storage pump

with the complete equipment as used in actual measurements.

The strip must be carefully stuck on a smooth surface which has first been polished with fine emery paper. It is then allowed to dry, after which it is ready for use. The simplicity of the method and the fact that no great preparations are needed help to explain the success of the strain gauge. Measurements in which 500 or more gauges are applied at the same time are no rarity today, though such numbers would have been quite impossible with the old mechanical strain-measuring instruments.

The measuring principle is very simple when applied to static stress. The gauge is connected to one branch of a sensitive Wheatstone bridge (Fig. 4), while another unloaded gauge of the same design is connected to the other branch in order to compensate for any effects of temperature. This dummy strip is stuck on a piece of the same material in the neighbourhood of the point of measurement, so as to preclude any inaccuracy in the results due to thermal expansion.

When variable or pulsating stresses, such as vibrations or periodically varying loads, have to be measured, one of a number of electronic methods may be adopted. The stress variations are usually reproduced on the oscillograph screen, and can be photographed if necessary.

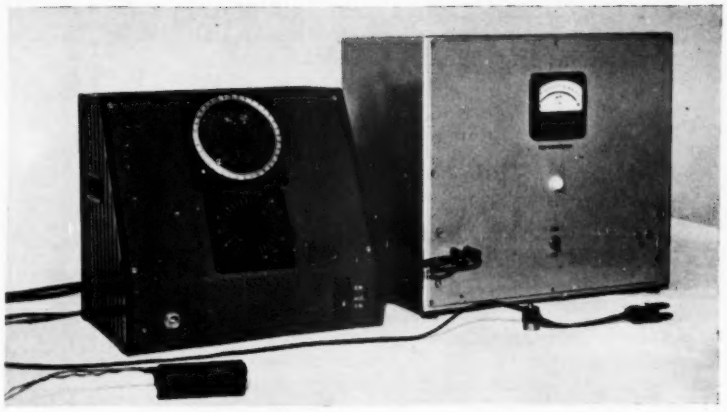


Fig. 6. Wheatstone bridge and balancing instrument. The electronic galvanometer was designed and constructed by Sulzer Brothers

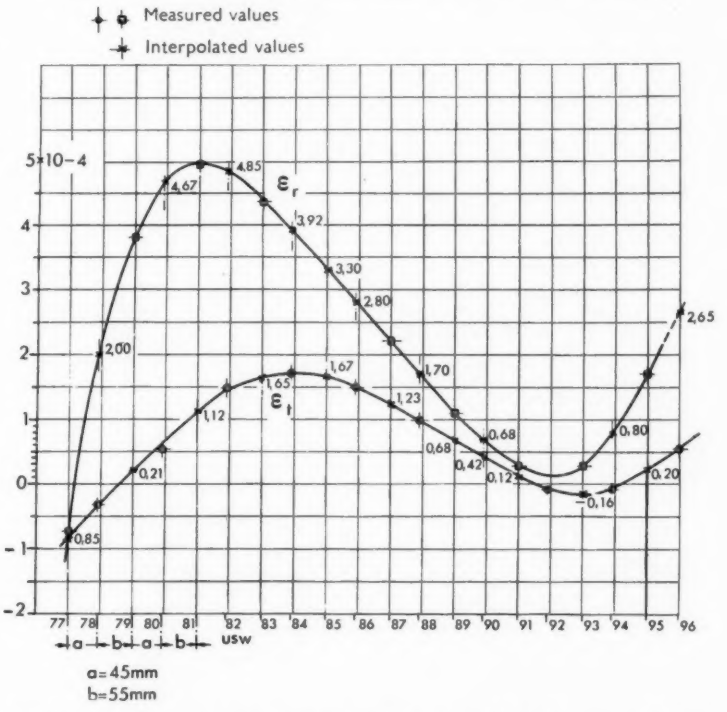


Fig. 7. Distribution of the strain over the most highly stressed cross-section of the discharge volute (quarter section)

Applications to Storage Pumps

Since the introduction of strain gauges, Sulzer Brothers have used them in numerous tests on pump casings and covers. These have furnished valuable information for the calculation and design of the main cross-section of high-lift components.

The storage pump for the Grimsel-Oberaar power station (Fig. 1), with which we are primarily concerned here, was designed and calculated on the basis of data obtained in earlier tests. The purpose of the measurements made on it was therefore not so much to investigate fundamentals as to prove the correctness of the calculations and of the general design of the pump. It was also desired to demonstrate the favourable effect of a few secondary modifications of the design. Finally, it was necessary to check one of the stipulations of the contract, which required that the stress in any part of the discharge casing at a test pressure of 105 kg. per sq. cm. abs. (about 1,500 lb. per sq. in. abs.) should not exceed one half the yield strength of an integrally cast specimen.

The test programme embraced the pump covers with suction branches and the discharge casing proper with the volute. Sections were chosen to cut the points of highest stress (Fig. 5) so that the longitudinal and transverse stresses could be measured along them. To throw more light on the mechanism of loading, other measurements were made at places outside the zones of highest stress. In particular, the effect of the ribs was investigated, as this is not easily amenable to calculation. When the surfaces had been prepared the gauges were applied lengthwise and crosswise in succession at suitable intervals along the chosen sections. as the measurement of the strain in the two principal

planes is essential for accurate determination of the stresses. The test programme called for more than 120 strain gauges. This large number, with the distribution of the gauges over the extensive surface of the whole pump, made it impossible to wire every single gauge to the measuring equipment in the usual way. This would have required at least 4,000 ft. of wire, and over 500 joints would have had to be soldered. As measurements with several hundreds of strain gauges to a single object are fairly frequently undertaken by Sulzer Brothers, a contact terminal has been developed to permit the speedy and reliable connection of the gauges to the measuring apparatus. The tips of the terminals are gold-plated to ensure good contact. The determination of strain by this method involves precision measurement of the electric resistances. The smallness of the changes in the resistance is the one difficulty of the method and makes extremely sensitive equipment and accurate work essential.

The strain gauges commercially available are designed so that a strain of 0·005 per mille—which represents a change in stress of about 0·1 kg. per sq. mm. in steel—corresponds to a change of 1/100,000th of an ohm. If the stresses are to be measured to an accuracy of 0·1 kg. per sq. mm. ± 10 %, the resistance measurements must therefore be accurate to one millionth of an ohm. To permit the reliable measurement of such small changes in resistance, the Sulzer physical laboratory has developed measuring equipment consisting of a calibrated measuring bridge and a very highly sensitive, electronically operating galvanometer. Both of these instruments have been constructed in the Sulzer laboratories (Fig. 6).

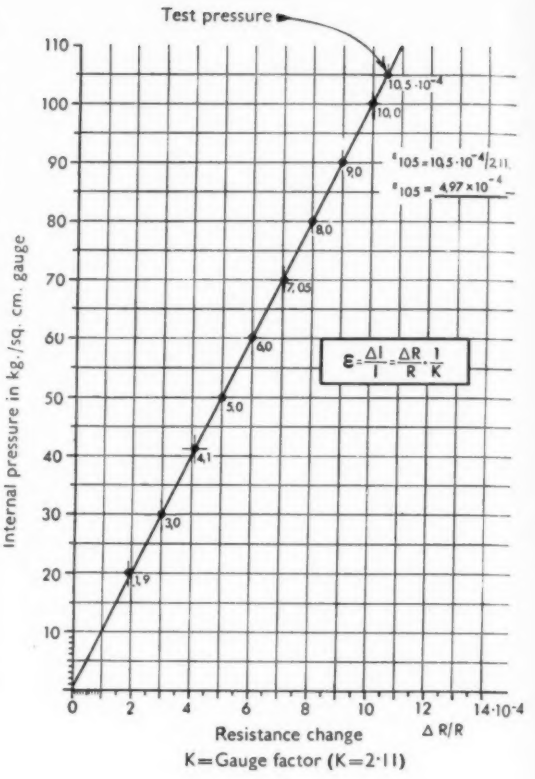

Test pressure
Internal pressure in kg./sq. cm. gauge
Resistance change
Δ R/R
K=Gauge factor (K=2·11)
$\varepsilon = \frac{\Delta l}{l} = \frac{\Delta R}{R} \cdot \frac{1}{K}$
$\varepsilon_{105} = 10{,}5 \cdot 10^{-4}/2{,}11$
$\sigma_{105} = 4{,}97 \times 10^{-4}$

Fig. 8. Change in resistance at the point of maximum stress. The diagram shows the change in resistance and thus the strain to have been absolutely proportional in the range of measurement

Measurements

The strain at all the points of measurement was determined at the test pressure of 105 kg. per sq. cm. Fig. 7 shows by way of example the curve for the

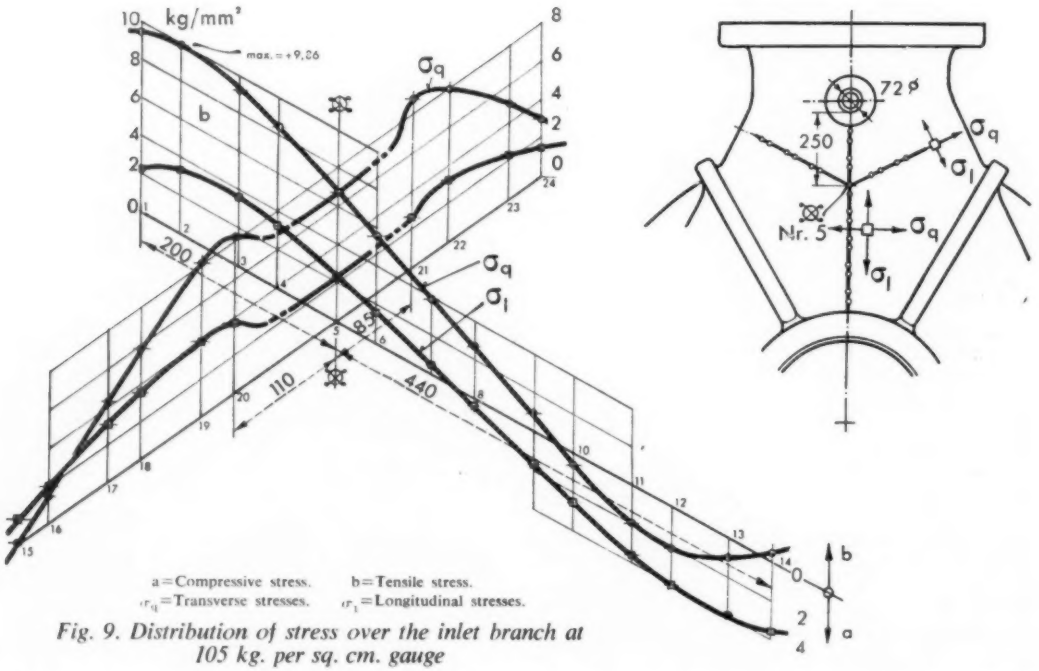
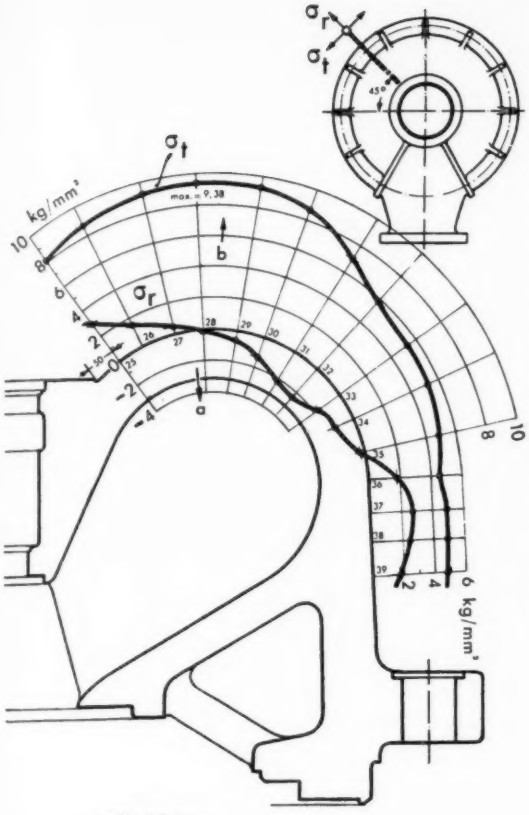


Fig. 9. Distribution of stress over the inlet branch at 105 kg. per sq. cm. gauge



σ_r = Radial stress.
σ_t = Tangential stress.
a = Compressive stress.
b = Tensile stress.

Fig 10. Distribution of stress over the inlet cover at 105 kg. per sq. cm. gauge

strain at the most heavily loaded section of the volute casing. In Fig. 8 the variation of resistance in the gauge applied at the point of maximum stress is plotted. It is clear from this that the deformation of the body of the pump was absolutely proportional up to the test pressure.

To determine the stresses at the surface of a component, the strains in the planes of the two principal stresses at right angles to each other must be known. These stresses σ_1 and σ_2 are related to the corresponding strains ϵ_1 and ϵ_2 by the following equations:

$$\sigma_1 = \frac{\epsilon_1 + \mu\epsilon_2}{(1 - \mu^2)} \cdot E \qquad \sigma_2 = \frac{\epsilon_2 + \mu\epsilon_1}{(1 - \mu^2)} \cdot E$$

where μ is Poisson's ratio and E is Young's modulus of elasticity. For purposes of evalution, the strains measured at all the sections were taken from graphs of the form illustrated in Fig. 7. To save time-consuming computations, a nomograph was used, with the aid of which the stresses for every point of measurement were calculated and plotted in the usual way. Figs. 9 to 12 show a number of the stress curves for the most important points of the pump casing.

Test Results

The smooth and gentle course of the stresses along the sections at once catches the eye. This points to a good choice of sectional gradations and wall thicknesses and proves the main components to have been well designed. The conclusions drawn from previous strain measurements on similar pump components are thus shown to be basically correct.

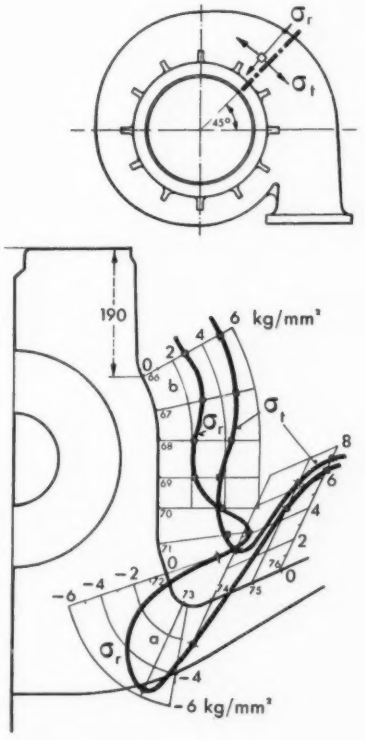
A particularly striking feature is the smoothness of the stress curve for the suction cover (Fig. 10). The choice of the sections, particularly in the region of the stuffing-box bore, and the subdivision of the large cross-sections in the flange have here permitted the stress to be kept at an approximately uniform level over a wide range, without any sudden variations. The result is a component which, with its smooth changes of cross-section and its simple form, is also very well adapted to the requirements of the foundry.

The greatest stresses occur in the discharge volute at point 81 (Fig. 12). The figures are as follows:

Radial stress = 11·8 kg. per sq. mm.
Tangential stress = 5·3 kg. per sq. mm.

As the lowest yield strength measured in an integrally cast specimen was just 25 kg. per sq. mm., the contractual stipulations were fulfilled with a generous margin.

It is of course not strictly permissible to make a direct comparison between biaxial stresses and the yield point established in the uniaxial tensile test. Numerous experiments have shown that plastic flow in forged or cast steel begins only after a certain



σ_r = Radial (and longitudinal) stress.
σ_t = Tangential stress
a = Compressive stress
b = Tensile stress.

Fig. 11. Distribution of stress at the junction of volute and main casing at 105 kg. per sq. cm. gauge

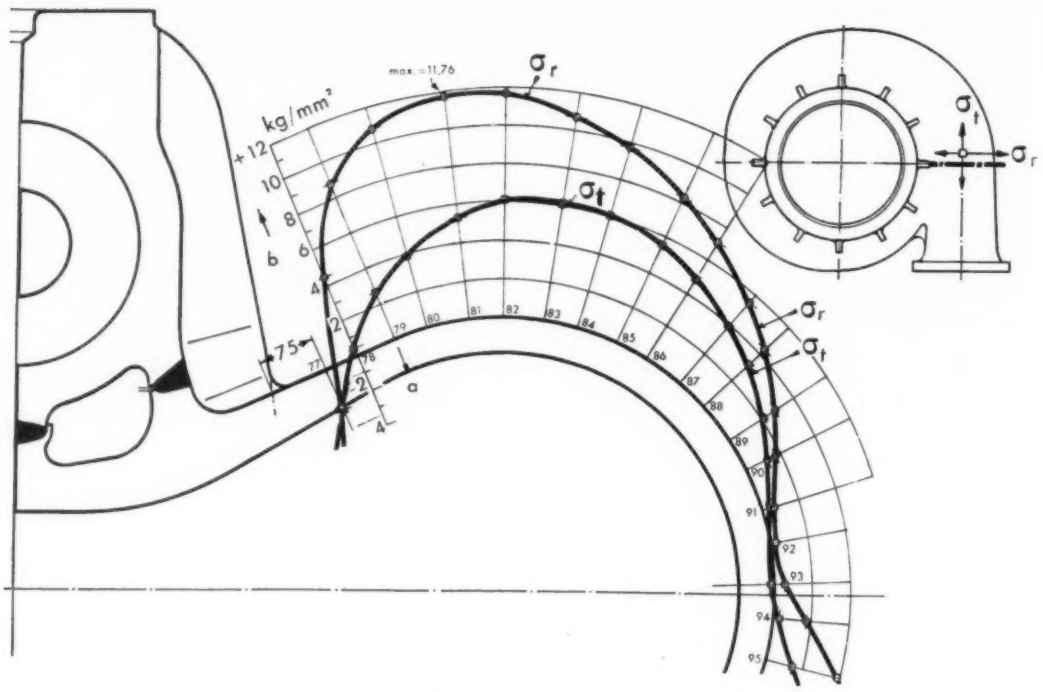


Fig. 12. Distribution of stress at the most highly stressed cross-section of the volute at p_i = 105 kg. per sq. cm. gauge

"equivalent simple tensile stress" has been reached, which can be calculated by the theory of constant shear-strain energy. The true safety of a component subjected to biaxial stressing, as in the case in point, can be established only by calculating this equivalent simple tensile stress σ_v:

$$\sigma_v = \sqrt{\sigma_1^2 + \sigma_2^2 - \sigma_1\sigma_2}$$

The equivalent stress figure thus obtained can be directly compared with the strength measured in the uniaxial test.

The maximum stresses occurring in the present case correspond to a maximum equivalent simple tensile stress of σ_v = 10·2 kg. per sq. mm. As the yield point is 25 kg. per sq. mm., the safety factor against plastic flow can be calculated as 25/10·2 = 2·45. This safety factor is more than sufficient, and may be classed as very high. The specific requirements were thus also fulfilled in respect of multi-dimensional stressing, and with a good margin in hand.

B.S.I. Annual Report

The Annual Report of the British Standards Institu- for the year ended March 31, 1955, contains in its 240-odd pages a review of the very wide range of activity which the Institution now embraces. The introductory "General Report" refers to two fields—Codes of Practice and Consumer Goods—in which the B.S.I. has recently extended its activities and for which two new Councils have been set up. A new section of the Divisional Reports is devoted to the work of the Council for Codes of Practice, which has now been in action for over a year. The Advisory Council on Standards for Consumer Goods has only been in effective action for a few months and has not as yet issued a report on its work, but the background to its creation and the problems with which it is to deal are set out in the "General Report." The accounts of international committees, contained in a separate section, and of the overseas activities of the Institution generally, reflect the increasing importance of this aspect of standards and its claims on the resources of the Institution.

The largest section of the report is naturally concerned with the projects for British Standards which have been under consideration during the year by some 60 major industries. This section includes notes on some of the more important standards issued during the year and on significant developments which have taken place on other projects. The number of projects in hand at any time is, of course, so great—totalling many thousands—that only a selection can be described or commented on in the report, but complete lists are given under each main committee.

A page of statistics, included in the "General Report," indicates at a glance the continuing scale of B.S.I. activity—just under 900,000 copies of standards sold in the year, over 4,000 meetings of committees held, 1,300 licences to use the B.S. Certification Marks issued. The number of subscribing members of the Institution is now 8,300 drawn from commercial firms, trade associations, and professional bodies.

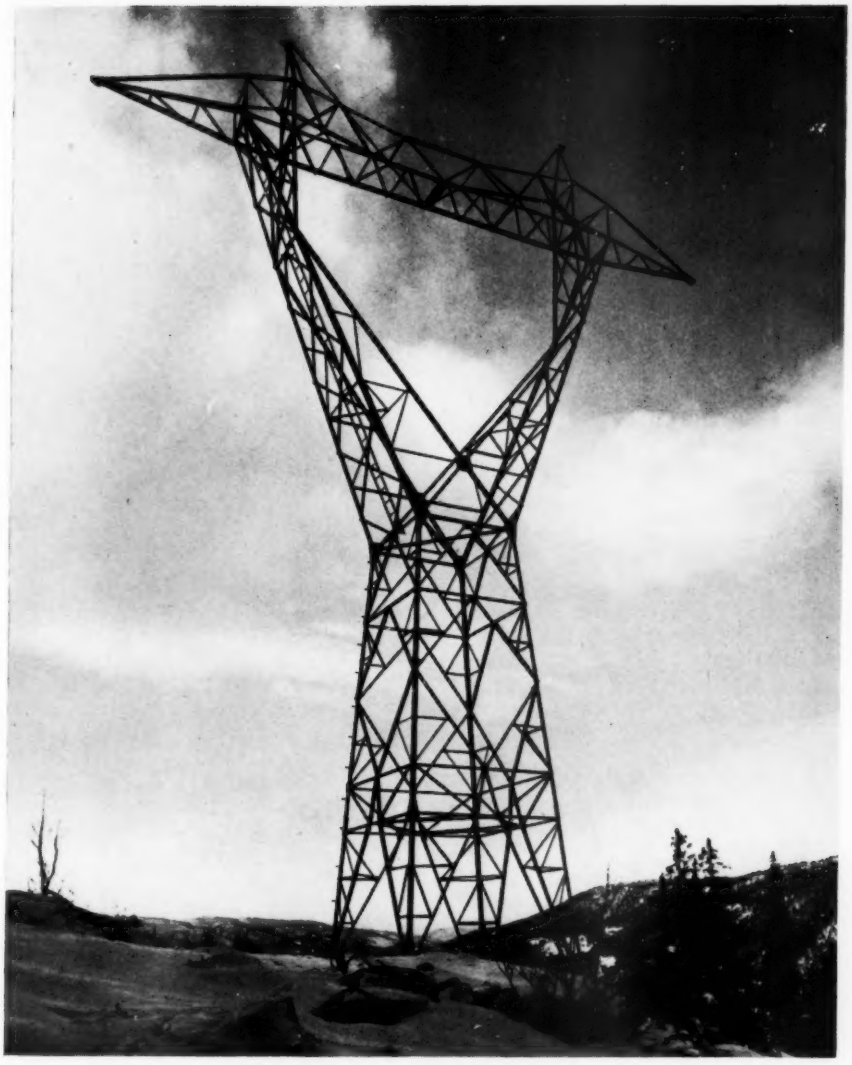


Fig. 23. A Blaw-Knox light suspension tower erected on site

The Hol Developments

In this final article we describe the 220 kV transmission lines to Oslo, and outline the Hol II and Hol III developments now in course of construction

PART THREE

AT the time of its construction the first transmission line from Hol to Oslo was the longest high-voltage line in Norway and the first to operate at 220 kV, the highest voltage used hitherto having been 132 kV. Other 220 kV lines are now constructed from Mår and Vinstra to Oslo, and these lines will form the basis of a 220 kV grid.

The Hol-Oslo line is 186·5 km. long and runs through Hallingdal on to Lake Krøderen, across the Leknes Hills to the Sokna Valley, passes to the north

of Hønefoss, and thence runs through the Langli River basin to Sogn substation. To reduce electrical interference with telephone and telegraph services the transmission line has been kept as far away from these low-energy lines as practicable, and this has involved routing it for long stretches over high ground. To reach the construction sites about 150 km. of road or track had to be built, and material for about 50 of the towers had to be transported by ropeway. For a total distance of 167 km. the line runs through forest land, and a belt 45 m. wide has had to be cleared.

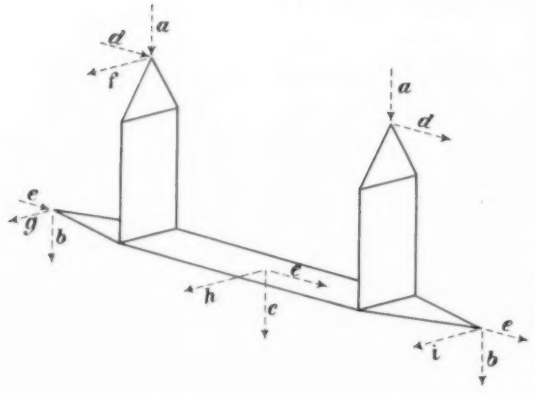
In this first line are 640 towers, of which 612 are of steel, built by the Bethlehem Steel Company, U.S.A., the total tonnage of steel, including foundation reinforcement, being 5,400 tons. The remaining 28 towers were built experimentally in reinforced concrete but proved to be more expensive than the steel towers. For

Fig. 24. Part of the first 220 kV transmission line, incorporating Bethlehem Steel Company's towers

TABLE II. TEST LOADS ON BLAW KNOX TOWERS (LB.)

Tower Type	Case No.	Test No.	Vertical			Transverse		Longitudinal			
			a	*b*	*c*	*d*	*e*	*f*	*g*	*h*	*i*
BM.I Light Suspension	1	1	7,337	12,827	12,827	—	—	—	—	—	—
		2	7,337	12,827	4,008	—	—	—	—	—	—
		3	7,337	4,008	12,827	—	—	—	—	—	—
	2	4*	1,274	4,151	4,151	1,314	4,806	—	—	—	—
	3	5	3,880	7,176	7,176	—	—	10,582	—	—	—
		6	3,880	7,176	7,176	—	—	—	—	14,991	—
		7	3,880	7,176	7,176	—	—	—	14,991	—	—
VFM.I 15°—40° Angle Dead End	1	1	—	—	—	10,858	15,381	—	—	—	—
		2	7,337	13,091	13,091	10,858	15,381	—	—	—	—
		3	7,337	13,091	4,273	10,858	15,381	—	—	—	—
		4	7,337	4,273	13,091	10,858	15,381	—	—	—	—
	2	5*	—	—	—	12,169	21,171	—	—	—	—
		6*	1,274	4,548	4,548	12,169	21,171	—	—	—	—
	3	7*	7,337	13,091	13,091	10,858	17,454	22,487	31,856	—	—
		8*	7,337	13,091	13,091	10,858	17,454	—	37,478	37,478	—
		9*	7,337	13,091	13,091	10,858	17,454	—	37,478	—	37,478
		10*	7,337	13,091	13,091	10,858	17,454	—	—	22,487	31,856

*Additional loads were applied at various points representing wind on tower at 25·6 lb. per sq. ft.



design purposes the route was divided into two zones dependent on the degree of exposure. In the lightly loaded zone, extending for 152·9 km., the suspension towers are designed for an unbalanced load of 50 per cent. of the ultimate strength of the cables; for the remaining 33·6 km. constituting the heavily loaded zone, the considerably higher loading of 80 per cent. of the ultimate cable strength has been deemed necessary.

Steel-cored aluminium conductors are used, of 380 sq. mm. cross section, equivalent to 240 sq. mm. of copper. They are suspended three in line on single strings of 18 porcelain insulators.

The New 220 kV Line

The second 220 kV line to Oslo, which will be needed when the third and fourth generating sets are

in commission, is now under construction and will follow the route of the existing line; for this reason the track is being cleared to a total width of 75 m.

The contract for the design and supply of the towers was awarded to Blaw Knox Limited, London, who have been able to apply the experience they have gained in their work for the British 275/380 kV supergrid, as a result of which the new towers, although designed for heavier loading conditions than those for the existing line, are between 20 and 30 per cent. lighter in weight. This has been achieved by the use on an average of 50 per cent. of high-tensile steel to B.S.548, the remainder being mild steel to B.S.15.

A further advantage of the use of high-tensile steel is that the base width of the new towers is only five-eighths of that of the existing ones. Consequently hillside frames are simpler and cheaper, and for most of the towers it will be possible to set out the foundations by means of an aluminium-alloy template frame. The smaller base width naturally leads to heavier foundation loadings, but a new technique of foundation design has been used which dispenses with reinforcement and considerably reduces foundation cost.

An interesting detail in the design is the method of slinging the earth wires. These are specified to be suspended above the power lines but midway between them so that they will not foul the power lines in the event of a break or a stretch due to icing. This has been accomplished by a simple triangular framework carrying a short overhanging swivel bracket which enables the wire to be strung without threading through the tower structure.

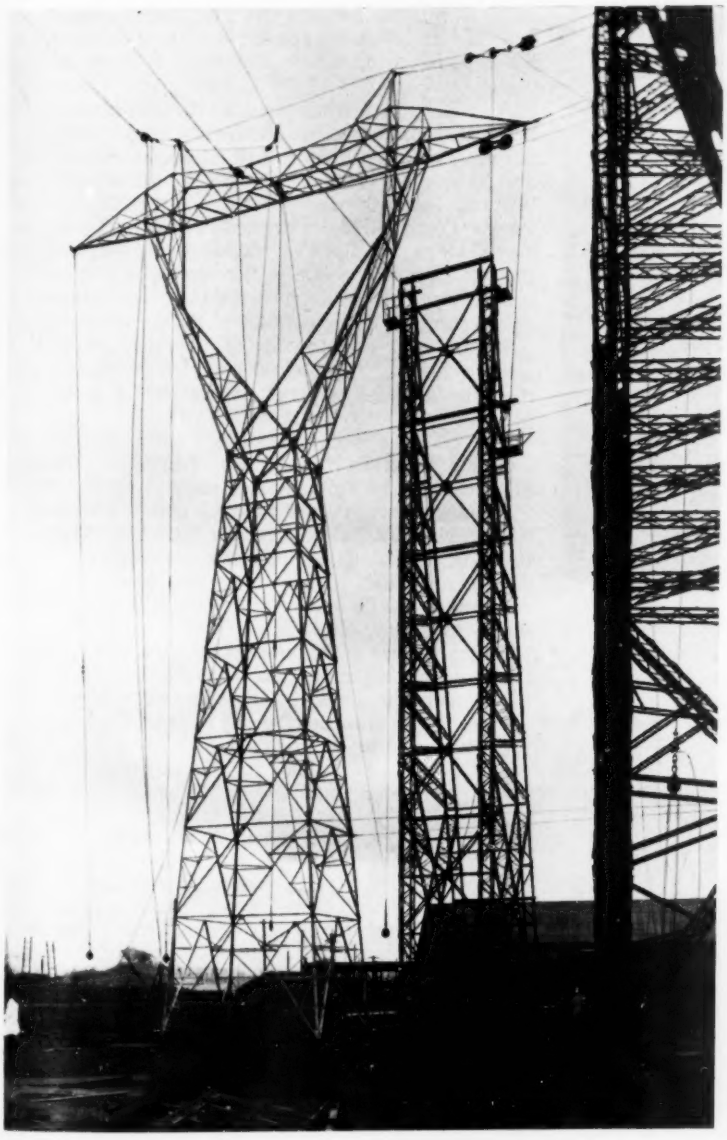


Fig. 25. One of the Blaw-Knox light suspension towers on test at the works of Painter Bros. Ltd.

Half of the towers are being fabricated in England by Painter Bros. Ltd., and the remainder in Norway by Teensberg Harpunfabrik A/S, who have built a new extension to their works to handle the order.

One of the light suspension towers under test at the Hereford works of Painter Bros. Ltd. can be seen in Fig. 25, and the performance of the towers can be judged from Table II giving particulars of the loads actually sustained on test. The weights of the towers, including foundation steelwork, are as follows:

Light Suspension Tower—Zone 1
29 m. high (as test tower) - - 6·75 tons
41 m. high (as test tower) - - 10·35 tons
Heavy Angle Tower—15°-40°
17·5 m. high (as test tower) - 9·75 tons

One of the light suspension towers under construction on site is depicted in Fig. 23 and a heavy angle tower in Fig. 26.

Sogn Substation

At Sogn substation, near Oslo, the two 220 kV lines from Hol will be received on a double-busbar system, and the power will be stepped down to 30 kV by two A/S Per Kure transformer banks. Each bank is rated at 120 MVA and consists of three single-phase units connected star-delta. Each unit has a transportation weight of 60 tons and a weight in service condition of 103 tons. A third winding is provided to give a voltage of 10·75 kV (when star-connected) for a 40 MVA 600 r.p.m. Westinghouse synchronous condenser for each bank. The total guaranteed losses at full load for each machine are 710 kW; the subtransient reactance is 18 per cent., the transient reactance 33·5 per cent. (unsaturated 38·5 per cent.), and the synchronous re-

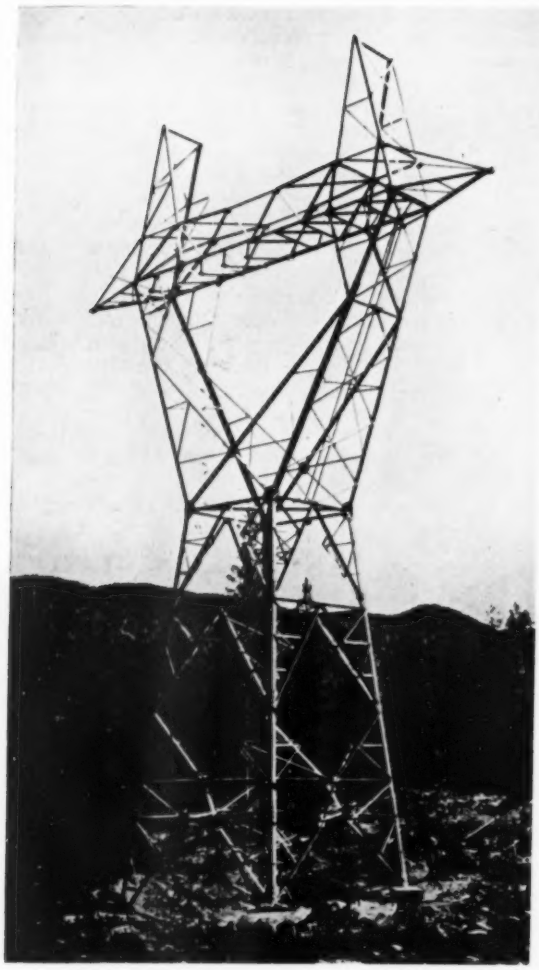


Fig. 26. A Blaw-Knox heavy angle tower erected on site and awaiting stringing

Fig. 27. General view of the buildings of Sogn substation near Oslo

actance 188 per cent. Each machine has a 140 kW 250 V exciter and a starting motor.

Hol II

As mentioned in our previous article, two further developments, known as Hol II and Hol III, are under construction downstream of Ruud power station. Hol II, layout drawings of which appear in Figs. 29 and 30, will replace the temporary station at Djupedal, and will utilise a head of 48 m. derived from the Djupedal falls, lying between Hövsfjord and Holsfjord. When the Djupedal station was built, a low overfall weir was thrown across the outlet from Hövsfjord; this weir will now be equipped with gates and the fjord used for daily regulation. The surface area of the fjord is about 1 sq. km. and a fluctuation in water level of about 1 m. will be allowed. The average regulated flow will be 30 cu. m. per sec. and the new station is designed for a maximum discharge of 70 cu. m. per sec.

As will be seen from the drawings the station will be located underground. The intake tunnel, including the pressure-shaft section, will be about 400 m. long and 50 sq. m. in cross section, the low-pressure section being unlined but blasted with trimming shots in the manner described in our previous article, and the pressure-shaft section lined. A trashrack will be placed at the intake, a sector gate in the surge chamber at the commencement of the pressure-shaft section, and a debris sump at the lower end of the pressure shaft.

Immediately downstream of the machine hall is a surge chamber, forming the commencement of a 600 m. unlined tailrace tunnel beyond which a 450 m. open channel leads to the river.

The machine hall will house a single unit, which will be a Kvaerner Brug/NEBB 33 MVA set generating at 12 kV, the voltage being stepped up to 220 kV by a transformer in an adjacent chamber. For the first time in Norway, bare busbars at 220 kV will be used to bring the supply to the surface, these bars being carried in a shaft 50 sq. m. in cross section. Thence connection will be made to the Hol-Oslo transmission line, which runs immediately over the site.

When we visited the development, the adit shown in the plan (Fig. 30) leading to the intake tunnel had been driven and the workings opened out, and excavation for the intake structure was in hand. Considerable progress had also been made with the main access tunnel to the power house. To minimise disturbance to the rock roof of this tunnel by blasting, the lower half of the tunnel was driven first and then the top half was taken out on the retreat. By this means it was possible to cut the roof with comparatively light parting shots. To secure the tunnel portal use has been made of rock bolts inserted into the face and sides. These bolts range from 2 m. to 4 m. in length and are fitted with wedge anchors.

Hol III

Hol III, which is also in an early stage of construction, will utilise a head of 97 m. between

Holsfjord and Strandefjord. Holsfjord, which has an area of 1·3 sq. km., will have a low overfall weir thrown across its outlet. The average regulated flow at the power-station intake will be 31 cu. m. per sec., and the plant is designed for a maximum flow of 75 cu. m. per sec.

The machine hall will lie at the end of a 4 km. unlined intake tunnel, leading to a surge chamber from which two lined pressure shafts will descend to the machine hall. Thence a tailrace tunnel of 0·75 km. long will lead to Strandefjord through a 0·5 km. open channel.

An interesting feature of the scheme is the use of a by-pass channel, as seen in Fig. 31. The main purpose of this channel is to maintain a flow in the winter months during periods when the power station is not running and thus to prevent freezing up. In addition it provides an overflow for the surge shaft, it provides a path for the clearance of tunnel debris, and it enables the supply tunnel and surge shaft to be emptied for inspection without disturbing the station.

Two 35 MVA Francis sets are to be installed. Each will be connected to its own transformer bank in an adjacent chamber, and, as at Hol II, the 220 kV supply will be brought out through a vertical shaft on bare busbars. Here, however, a crosscountry line about 1 km. long will be required to join up with the Hol-Oslo line.

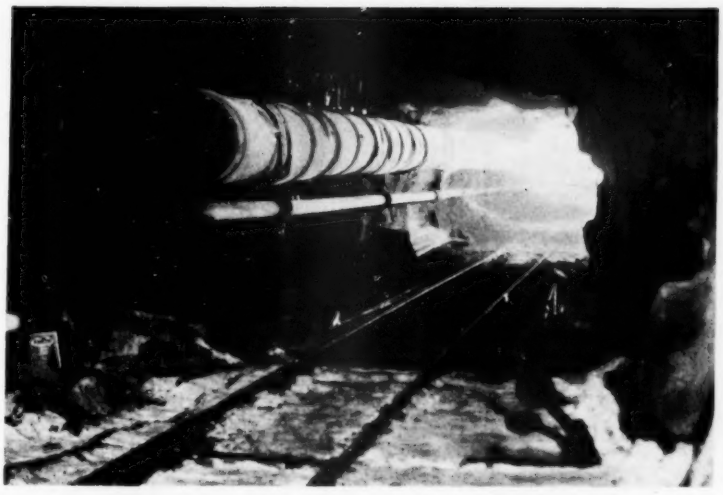


Fig. 28. Adit No. 2 of the supply tunnel for the Hol II development

The main civil-engineering contractors for both Hol II and Hol III are A/S Høyer Ellefson. Atlas Diesel drilling equipment and LM 200 shovels are being used for tunnelling, together with Ruston diesel locomotives and Euclid trucks.

Manufacturers and Contractors

The main civil-engineering contractors for the Hol developments are A/S Høyer Ellefsen and Civ. Eng. Thor Furuholmen, whereas the building of Sogn substation was handled by Fagbygg A/S. The following is

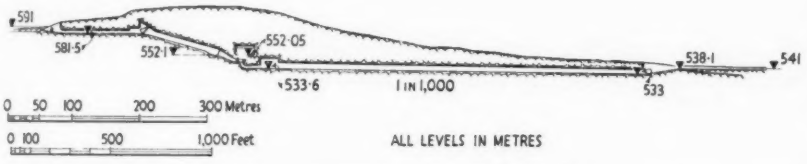
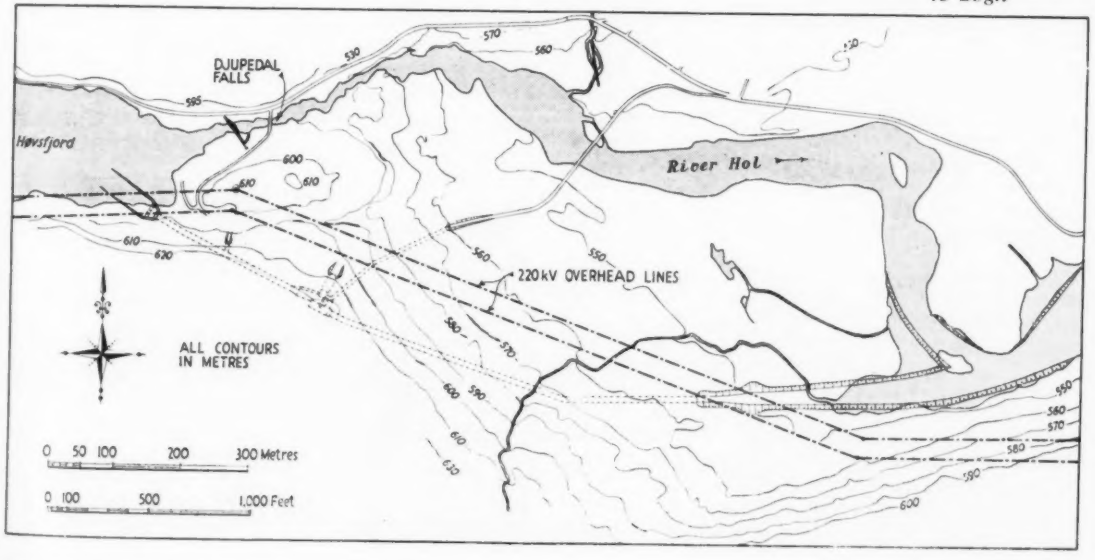


Fig. 29 (left). Profile of the Hol II development

Fig. 30 (below). Plan of the Hol II development showing the route of the 220 kV lines from Hol I to Sogn



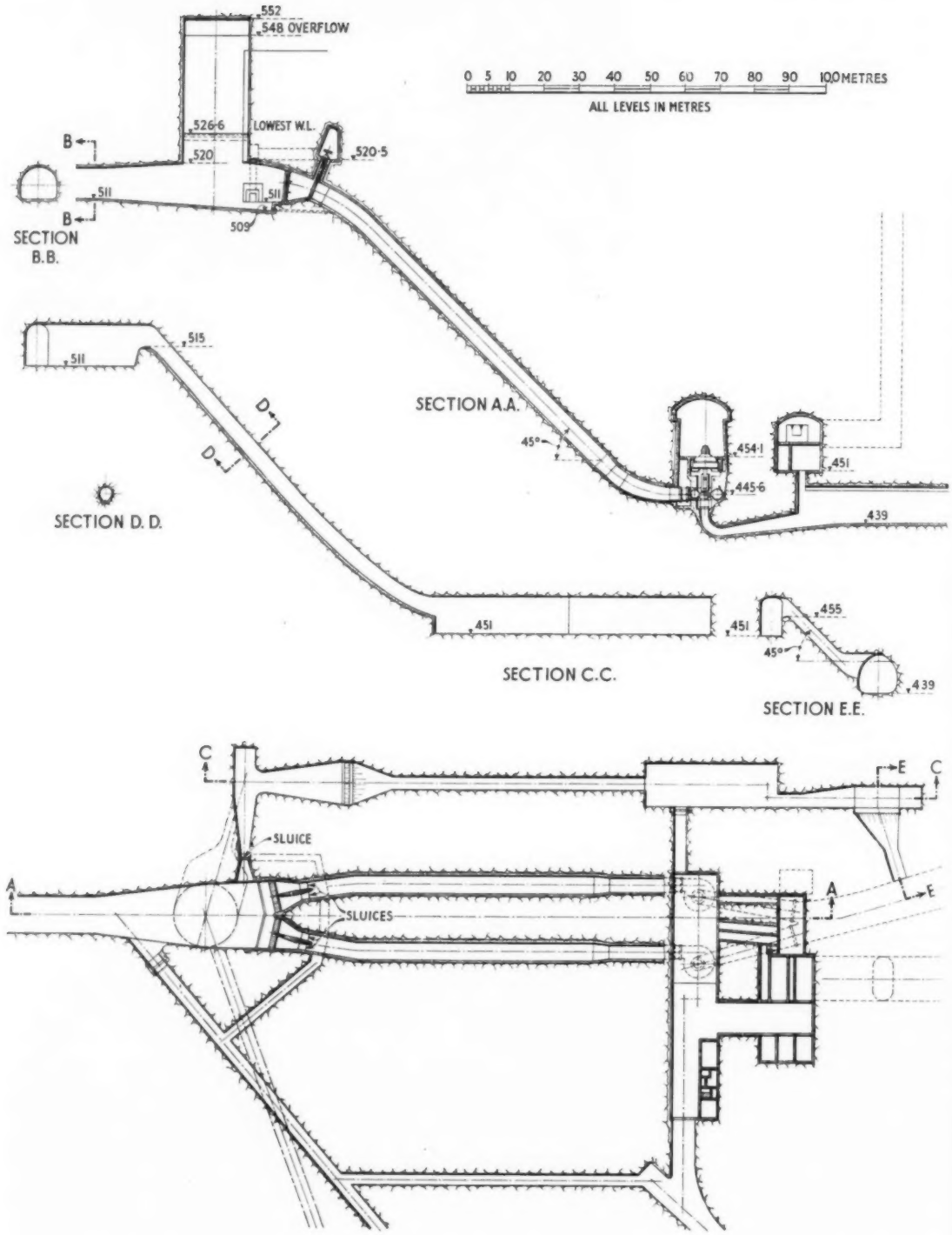


Fig. 31. Sectional elevation and plan of Hol III power station and associated works

a list of the principal manufacturers concerned:

Penstocks: Etabl. Bouchayer & Viallet and A/S Kvaerner Brug. Erection of the French sections by Erik Ruude Mok, Verksted. Turbines, Sluice Valves and Relief Valves: A/S Kvaerner Brug. Cranes: *Ruud, Hol II*, S. H. Lund & Company, Oslo; *Sogn, Hol III*, A/S Drammens Jernstøperi. Generators, Generator Switchgear and Instrument Transformers, and Control and Protective Gear: A/S Norsk Elektrisk & Brown Boveri. 220 kV Circuit Breakers and Instrument Transformers: Brown Boveri & Co. Ltd., Baden. Transformers: *Ruud*, A/S National Industri; *Sogn, Hol II, Hol III*, A/S Per Kure. First 220 kV Transmission Line: *Towers*, Bethlehem Steel Company; *Insulators and Fittings*, Ohio Brass Company. Second 220 kV Transmission Line: *Towers*, Blaw Knox Limited. Synchronous Condensers at Sogn: Westinghouse Electric Company.

Acknowledgments

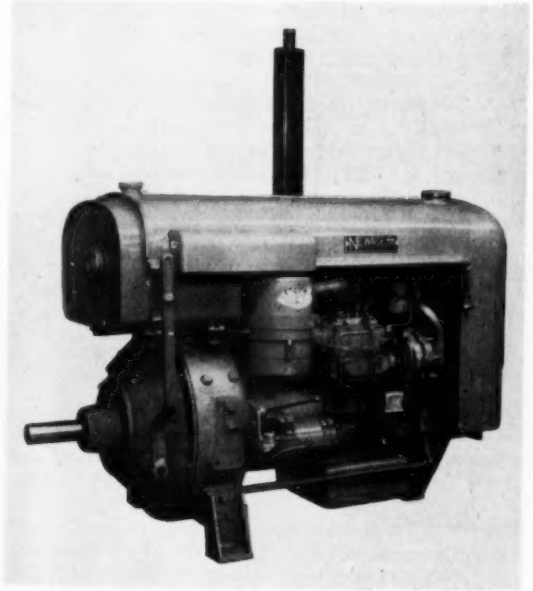
In publishing these articles we have much pleasure in acknowledging the courtesy of Mr. Erling Eriksen, President of Oslo Lysverker, and of Mr. L. Ljøgodt, Director of Planning and Construction, in according us permission to visit the Hol developments and in placing information, drawings and photographs at our disposal. Amongst Mr. Ljøgodt's immediate staff, his deputy, Mr. Hj. Mjelstad, Chief Engineer Mechanical and Electrical Department, and Mr. H. P. Storebø, Chief Engineer Transmission Lines, have been most helpful, and we were much indebted to Mr. A. Berg, Resident Engineer at Hol, for his courtesy and helpfulness during our inspection of the developments. Much useful information was given to us during visits to the works of A/S Kvaerner Brug and A/S Norsk Elektrisk & Brown Boveri, and we derived much assistance from an article on the Hol developments in "Norske Kraftverker," the centenary volume produced by *Teknisk Ukeblad*, the Journal of the Norwegian Society of Civil Engineers, which we reviewed in our September 1954 issue. Finally we should like to express our appreciation of the invaluable assistance afforded to us by Mr. R. Heggstad, Chief Engineer of Kraftforsyningens Sivilforsvarsnemnd, Oslo, and Mr. H. S. Jacobsen, formerly Managing Director of A/S Atlas Diesel, Oslo, in making all needful arrangements on our behalf and acting as "guides, counsellors and friends" throughout our tour.

Diesel Power Pack

A new diesel power pack suitable for driving all kinds of civil-engineering contractors' equipment—excavators, earth-moving equipment, crushing plant, concrete mixers, mobile cranes, compressors, generators, welding sets, etc.—has just been placed on the market by Newage (Manchester) Ltd., 6 Carlos Place, Grosvenor Square, London, W.1. It is built in two sizes, a 3·4 litre unit giving outputs ranging from 27 b.h.p. at 1,200 r.p.m. to 42 b.h.p. at 2,000 r.p.m., and a 5·1 litre unit with a corresponding range of 40 to 63 b.h.p.

Injection is by a Simms pump, and the governor is either pneumatic or centrifugal, a hand control being provided to set the running speed as desired. Lubrication is by high-pressure feed, and water cooling is by pump circulation through a radiator of temperate or tropical capacity. Lucas electrical equipment includes a 12 V battery, dynamo, and pre-engagement type starter. The drive is taken through a hand-operated Rockford clutch.

The power pack is self-contained and is carried on a three-point mounting. Particular attention has been paid to accessibility and to sturdy and reliable design, and world-wide service facilities are available. The accompanying illustration shows a 3·4 litre unit.



Riva Calzoni. Costruzioni Meccaniche Riva S.p.A of Milan, and their subsidiary, Alessandro Calzoni S.p.A., of Bologna, have jointly issued an extremely well-produced catalogue which presents an impressive conception of their works, the way in which they are equipped, and the quality and variety of goods they turn out. These are for the most part water turbines of all types and sizes but the Calzoni works also manufacture machine tools and pneumatic hand tools. The numerous illustrations, many of which are in colour, are captioned in both Italian and English.

Mobile Cranes. Two catalogues, printed in colour, have been received from Ransomes & Rapier Limited and describe respectively the "3 Standard" (Publication No. 495) and the "6 and 7 Standard" (Publication No. 1/494) mobile cranes. These can be fitted alternatively with diesel-electric or petrol-electric drives, and either solid or pneumatic tyres can be provided. The publications are copiously illustrated and full technical particulars are given, together with tables showing the ratings (British, American and metric), outreach, lift above and below ground, etc. All these cranes carry a three years' guarantee.

Abstracts from the World Technical Press

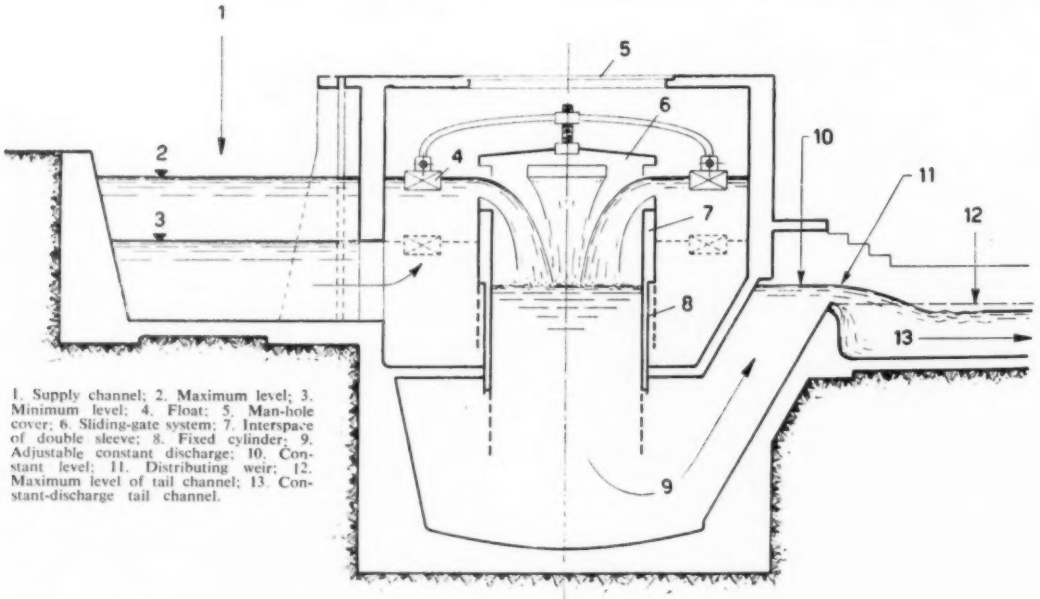
Automatic Constant-Discharge Regulator for Diversion Channels

Ing. Cogliati's automatic flow regulator ensures, it is claimed, a rigorously constant discharge at any predetermined rate in diversion channels fed from variable-level reservoirs or canals, and its working remains fully unaffected by level fluctuations occurring upstream or downstream from the diversion point. Roughly speaking, the design of this flow regulator presents some similarity with that of a two-chamber surge tank; it consists, in fact, of two chambers, one above the other, communicating by a central pipe extending inside the two tanks, which may be either circular or square in cross-section. The upper or flow-regulating tank connects with the supply reservoir or canal, while the lower tank has a siphon-like outlet with a distributing weir set up at the head of the tail channel; the connecting pipe is an open cylinder fixed on the periphery of the central opening of the diaphragm formed by the bottom of the upper, and the top of the lower chamber. Two coaxial cylinders, connected at the top by a welded distance ring, form a double sleeve which encases the fixed cylinder; a number of ports provided at their upper periphery allow the inflowing water to pour from the upper into the lower tank. On the other hand, the double sleeve is integrated with a float so as to constitute a kind of sliding-gate system which closely follows any level variation occurring in the upper tank. Stress is laid on the following features of the Cogliati flow regulator:—

(1) The relative position between float and ports can be accurately set by means of a simple adjusting gear; (2) the interspace in the double sleeve is wide enough to allow the mobile gating system full freedom of travel without any friction; (3) leakage of water through siphoning in the double sleeve is made impossible by a pneumatic gasket. This flow regulator can be incorporated in any hydro-electric, water-supply, irrigation or any other system in which a strictly constant discharge is required. A few figures are given at the end of the article expressing the discharge capacities for given cross-sections of the upper chamber; for instance, a tank with a diameter or square sides of 3·1 m. would pass a discharge of 500 litres per second whereas a diameter or square side of 1·55 m. would pass only 50 litres per second. (*Giornale del Genio Civile*, Vol. 93, May, 1955, No. 5, pp. 316/317, 1 f.)

The ANIDEL Annual Meeting

From the report of the ANIDEL Council, read at the 10th annual meeting held in Rome on July 22, 1955, we learn that the aggregate Italian production in 1954 was as follows: Hydro plants 29,217 million kWh; thermal plants 4,476 million kWh and geothermal plants 1,886 million kWh, i.e., a total of 35,574 million kWh. After deducting an excess of 333 million kWh of exports (646 million) over imports (312 million), the net production available was therefore 35,241 million kWh, which, compared with the 1938 figures, represents an increase of 230 per cent. The aggregate length of transmission lines was 25,221·8 km., comprising 6,064·4 km. of 220-230 kV, and 19,157·4 km. of 120-150 kV lines. Last year's new constructions and enlargements have added 387,355 kW to the capacity available at the end of 1953. The progress achieved during the first post-war decade is best illustrated by the total consumption of 29,416 million kWh in 1954 against 14,038 million in 1946, an increase of 109·55 per cent. During the same period, consumption per head rose by 101·08 per cent. from 304·3 to 611·9 kWh. For the first four months of this year, there is a production increase of 4·27 per cent. compared with the same period of last year. A schedule worked out by commercial electrical undertakings

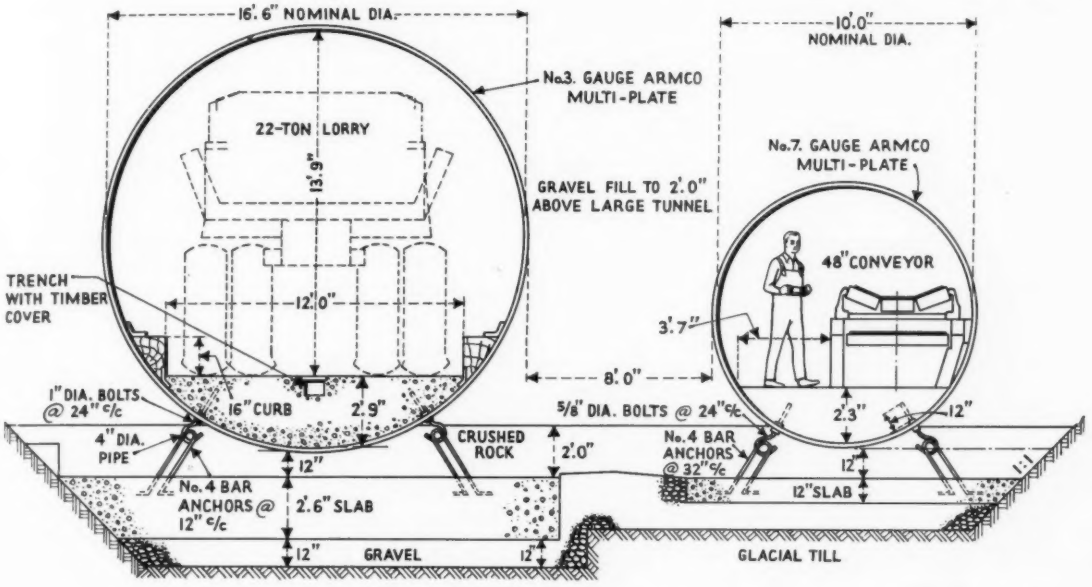


1. Supply channel; 2. Maximum level; 3. Minimum level; 4. Float; 5. Man-hole cover; 6. Sliding-gate system; 7. Interspace of double sleeve; 8. Fixed cylinder; 9. Adjustable constant discharge; 10. Constant level; 11. Distributing weir; 12. Maximum level of tail channel; 13. Constant-discharge tail channel.

at the request of the Ministry of Public Works provides, between 1955 and 1958, for the construction of new hydro and thermal plants which, upon completion, are to increase production by a further 8,000 million kWh. (*Quaderni di studi e notizie*, Vol. 11, No. 200/1, 1.8.1955, p. 563, 9 pp., 3 tables.)

Tunnel Access to St. Lawrence Project

Men and machines to build the Canadian half of Barnhart Island power house on the St. Lawrence have now tunnel access to their work. This article describes how two cut-and-cover corrugated-steel access tubes were laid beneath the Cornwall canal while winter had frozen shipping to a standstill. While the larger pipe, 16 ft. 6 in. inside diameter, will pass the large construction equipment on the job, the smaller, 10 ft. diameter pipe carries a 48 in. conveyor belt which leaves room enough for a pedestrian passageway. Tunnel foundations and anchorages against uplift appear in the drawings, as do pavements and a drainage trench in the larger tube. Gratings across the tunnel portals intercept surface water, and others at low points catch water for sump-pump collection and discharge. The tunnel length is 368 ft. portal-to-portal, and there is a 250 ft. approach ramp at each end. Tunnel interiors are painted and lighted, and vehicle traffic in the larger tube is controlled by a signal system. Shipping on the canal was reopened on its scheduled date of April 18, with both tubes completed and free of leaks. (*Engineering News-Record*, Vol. 155, No. 1, 7.7.1955, 2 pp., 7 ff.)



Storage Pumping

A brief analysis is given of the technical and economic problems involved in the setting up of hydro-electric plants combining generation and storage pumping. The author explains at the outset the methods he applied when ascertaining the cost of the various sections of such plants, including lower and upper reservoir, penstocks, and building and equipment of the power house. Installation and operation costs of a generating-pumping plant are compared to those of a thermal plant of similar energy output. When discussing location possibilities, examples are given of pumping plants co-ordinated with a master hydro-electric generating station operating without any reservoir, or with a series of mountain plants working from high heads or fed by natural lakes. These two solutions are particularly favourable, since the cost of the pumping plant per kW would be very small. In view of the economic advantages offered by the combination of hydro-electric developments with storage pumping, consideration should always be given to the possibility of such a combination when working out projects of chains of co-ordinated or synchronised power plants. (Zbigniew Zmigrodski, *Archiwum Hydrotechniki*, Vol. 1, No. 2, 1954, p. 167, 22 pp., 6 ff., 12 tables.)

Load Dispatching

The load-dispatching equipment put into operation a few months ago by the ATEL concern (Aar-Tessin S.A. d'Electricité) is described. In addition to the output of six plants of their own, ATEL have a substantial share in other hydro concerns, and all the energy distributed reaches through ATEL widely differing consumption centres and classes. The addition of the load dispatcher to the existing facilities (an extensive high-frequency telephone system and an up-to-date power-frequency governing equipment) now enables the management of the concern to exert a constant supervision of load distribution, and to intervene, when required, at the right time and right spot. The load dispatcher at the Olten operating centre consists of a diagram board reproducing the whole of the 150 kV and 225 kV ATEL systems, HT lines being represented by horizontal metal strips and the various bus-bar sets by vertical strips; a wattmeter is inserted in each line and shows not only the amount of energy but also the direction of the flux. Other wattmeters in-

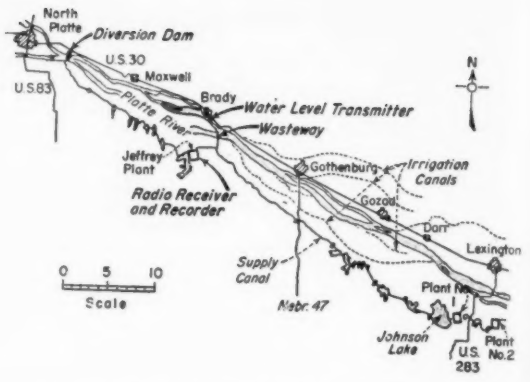
dicating the power transmitted from the Gösgen and Mettlen transformers as well as from the high-head Lucendro plant and Piottino regulating station, complete the distribution board. The Lucendro and Piottino instruments enable the Olten operating staff to supervise the utilisation of the power-water reserves stored in the reservoirs of these two plants. Among the direct-reading instruments, the varmeters of the Lukmanier line and of the three 100 MVA transformers at Mettlen are of particular importance, ensuring as they do the control of the amounts of reactive energy supplied by the Maggia concern to associated plants. Circuit breakers are represented at the end of each line by light-signal switches indicating clearly whether the breakers are open or closed. There is but one load supervisor at Olten during normal office hours; the 24-hour service continuity and full efficiency of the load dispatcher is ensured by an exact duplicate of the Olten board installed in the control room of the Gösgen plant. Clashing or overlapping of duties between the Olten supervisor and the Gösgen staff is checked by light signals controlled from the Olten operating room by a switch. (O. Zimmerli, head of ATEL Operating Centre, *Bulletin de l'Association Suisse des Electriciens*, Vol. 45 (1955), 21.7.1955, p. 657, 4 pp., 4 ff.)

Heavy Construction Plant in Switzerland

The report of the Second International Fair for Construction Machinery, held in Zurich from March 25 to April 3, 1955, gives a very favourable picture of the progress made by Swiss manufacturers in this special branch of engineering. Home production is expanding and, in not too far a time, may be in a position to satisfy most of the requirements of Swiss contractors, who are still depending to a large extent on imports, especially for heavy equipment. The report is divided into five sections: excavating and earth-moving equipment, machinery for the preparation and grading of materials (aggregates, sand, concrete), conveyors and vehicles, cranes and other handling plant, and finally tunnelling machinery and tools. (Ing. E. Bugmann, *Wasser- und Energiewirtschaft*, Vol. 47, No. 4, April 1955, p. 74, 4 pp., 7 ff.)

Power-Irrigation Co-Ordination by FM-Radio

This article describes the FM-radio control system adopted to ensure that the Platte River waters, Nebraska, meet both power and irrigation requirements in the best possible manner. Water to be controlled is diverted from the river just east of North Platte and carried down the supply canal through the Jeffrey power station to the wasteway, at which point it may be either returned to the river or sent down the canal to downstream power plants. The amount of water returned to the river depends on the flow at Brady and the daily fluctuating irrigation needs. Readings taken from a water-level gauge south of Brady are radioed to an automatic recorder at the Jeffrey plant, five miles away, and then passed on by power-line carrier to the hydrographer, by telephone to the wasteway-gate operator, or by two-way radio to the canal patrolmen, whereupon operator or patrolmen adjust the return flow to maintain the desired river level. Experience with remote water-level recording by radio has proved very satisfactory, and it is estimated that the value of the water saved in the first



season exceeds the costs of installing the equipment. (Edwin L. Hamilton, Central Nebraska Public Power & Irrigation District, *Electrical World*, Vol. 144, No. 8, August 22, 1955, p. 121 & 124, 3 ff.)

Turkish Irrigation-Power Scheme

An account is given of the progress of work at the Seyhan River Development, which will provide irrigation water for nearly 400,000 acres of the broad and fertile coastal plain near Adana, and supply with light and power the cities of Adana, Mercin, Tarsus and Ceyhan, now fed by Diesel generating systems. The Seyhan earthfill dam, on which work began in August 1953, was about half finished by early April 1955, and its spillway was complete, except for the control equipment which will consist of six radial steel gates discharging water into a concrete-lined channel leading to the river below the dam. Once completed, in the spring of 1956, 8 million cu. yd. of fill will have been placed in an embankment reaching 6,500 ft. across the river valley, and rising to a maximum 180 ft. About a mile upstream of the dam, on the right bank of the future reservoir, a second spillway channel, requiring 3,650,000 cu. yd. of excavation, is being carved; work is also well along at the dam left-end on the 185 × 100 ft. power house which is to be equipped with three 38,000 h.p. units and will be fed by three penstocks branching from the 2,500 ft. tunnel acting as river diversion during the dam construction. Flow through this 26 ft. diameter tunnel, also to be used for irrigation, will be controlled by a 200 ft. high concrete intake tower which had reached the 80 ft. mark in early April. (The *EM-Kayan*, Magazine of the Morrison-Knudsen Company, Vol. 14, No. 3, May 1955, p. 3, 3 pp., 7 ff.)

Note: This Journal also contains articles on the completed Box Canyon Dam (p. 6), and on the final construction phase at Chief Joseph Dam (p. 14).

CLASSIFIED ADVERTISEMENTS

Announcements for this column can be accepted up to the 8th of the month for the following month's issue. The charges are fourpence per word with a minimum of 10s. Box No. facilities 2s. 6d. extra. In order to avoid accountancy it would be appreciated if instructions to insert were accompanied with remittance.

Plant for Sale

Unused turbine by Gilkes, Kendal, with 11 KWS D.C. generator and switchboard, 230 volts, on 105 feet head with 48 Everite asbestos pipes and couplings, 9 in. bore (620 feet total) stop valve and 3 metal halfbends. Offers. Pearson, Abriachan, Inverness-shire.